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1930—1931.

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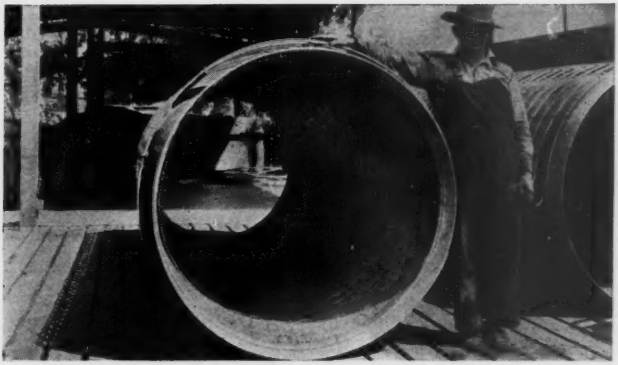

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New England Water Works Association
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THE NEW GRAVITY WATER-SUPPLY SYSTEM OF ALBANY, N. Y.

BY ROBERT E. HORTON* AND BENJAMIN L. SMITH.†

[*Read November 12, 1930.*]

Adoption and Economics of Project.

Looking toward a New Supply. Albany is located on the tidal section of the Hudson River 8 miles downstream from the head of tide at Troy and 143 miles from New York City. Settlement of the city was begun within a few years following the memorable voyage of discovery by Hendrick Hudson in 1608.

Albany has been the center, successively, of the fur trade, the lumber industry and the foundry and stove industry. Following the removal of the lumber and foundry industries to other centers, the growth of the city was retrograde for a time in the latter part of the nineteenth century. Beginning about 1910, the city has had a relatively rapid but normal growth and has a population, according to the United States Census of 1930, of 126 000. Including Troy, Schenectady and suburban cities and villages, there is a population of more than 350 000 within a radius of ten miles from a central point in the triangle connecting Albany, Troy and Schenectady. In spite of this concentrated population requiring water supply it happens that there are many sources from which Albany could obtain a suitable water supply for present and future requirements. The existence of so many available and suitable sources of water supply for the city has perhaps been a handicap. On the one hand it has created dissension and discussion over the merits of different supply sources covering a period of more than fifty years. On the other hand, it has led to a large amount of engineering work in the way of investigations and reports, so that a history of the Albany water supply reads like an excerpt from "Who's Who in Hydraulic Engineering."

Neglecting earlier local sources, an investigation was made by William J. McAlpine in 1850; as a result, Rensselaer Lake was constructed in the Pine plains northwest of Albany, with a 4-ft. brick conduit leading to the city. The gathering ground was a wilderness of permeable sand with a

*Consulting Hydraulic Engineer, Albany, N. Y.
†Of Whitman, Requardt and Smith, Engineers, Baltimore, Md., and Albany, N. Y.

sparse growth of needle-leaved trees and ideal for water-supply purposes. The drainage basin, however, was limited, so that while Rensselaer Lake still is an integral part of the Albany water-supply system, it is held in reserve, and an extension of the supply was soon required.

In 1869, the taking of water from the Mohawk River was considered in a report by S. H. Sweet, State Engineer, together with certain small streams. In 1872 James P. Kirkwood recommended taking a supply directly from the Hudson River, and this report was endorsed by Professor C. F. Chandler. The Quackenbush Pumping Station, taking water from the Hudson River, was put in operation in 1876 and is still in use in connection with the present filter plant. In 1878, John T. Fanning reported on the possibilities of supply for New York City and the Hudson Valley, including Albany, from Lake George and other Adirondack lakes.

In 1885, acting for a special water commission, N. E. Worthen investigated the use of the Hudson and Mohawk rivers, the Normanskill, Wynants Kill, lying east of the Hudson River, and a number of other streams. He finally recommended a small local stream supplemented by a driven-well supply. The attempt to obtain the latter was a failure. In 1885, Albert R. Leeds recommended the use of the Hudson River and another attempt to obtain a driven-well supply. Nothing further was done until 1891 when, acting for a new special commission, J. Herbert Shedd recommended the Normanskill, a tributary of the Hudson River to the west of the city, and this project was endorsed by John Bogart and Rudolph Hering. After a year's delay to obtain weir measurements of the Normanskill this source of supply was rejected as insufficient and because the quality of stored water was considered questionable. This last point is interesting in view of the now well-known fact that the quality of water improves with storage, under suitable conditions.

The Kinderhook was also considered and recommended. This is a stream lying east of the Hudson River. The Kinderhook project was endorsed by Clemens Herschel, J. J. R. Croes and Frederick P. Stearns. This proposition roused a storm of protest, the echoes of which are still heard in local politics.

In 1897, Allen Hazen recommended filtration of the Hudson River water. The use of slow sand-filters was advised because at that time rapid sand-filters were relatively new. The slow sand-filter plant was built under Mr. Hazen's direction in 1898, but with the growth of the city it soon became inadequate. Scrubbing filters were installed in 1908 with a view to increasing the effective capacity of the slow sand-filters. These were later converted into rapid sand-filters.

A serious typhoid epidemic was experienced in Albany following the flooding of the filter plant in the great flood of March, 1913. The direct pollution of the river from Troy, Schenectady and other upstream communities and by industrial wastes rendered the continuous, successful operation of the filter plant unusually difficult at the existing location.

Nevertheless, it may be said that the Albany filter plant is delivering water excellent in appearance and safe for drinking purposes. The difficulties of accomplishing this result under the existing conditions are, however, very great, and credit is due to George Willcomb, operating superintendent, and to Theodore Horton, advisory engineer on the operation of the filter plant.

By 1925, many conditions existed which demanded either the construction of a new filter plant at a new location or the adoption of a source of water supply other than the Hudson River. These conditions included:

1. The extreme difficulty of reliable, continuous and satisfactory treatment of the existing water owing to the location of the intake so short a distance below serious sources of pollution, with no appreciable chance for self-purification.
2. The necessity for an increased supply in any event.
3. The rapid growth of the city on the higher levels and the consequent existence of a large region in the better residential section of the city with inadequate pressure.
4. The insufficiency of the existing distribution reservoirs in the event of a serious breakdown or interruption of filter-plant operation, as by a flood.
5. The increasing aggregate cost of pumping and filtration, owing to the location and manner of construction, both of the pumping stations and filter plant, as the city gradually developed.

Fully cognizant of these facts, Mayor Hackett in 1926 engaged Nicholas S. Hill, Jr. to make a general reconnaissance and report on the available sources, including not only the Hudson and Mohawk rivers but others as well. Mr. Hill's report covers in a general way more than a dozen possible sources of supply for Albany which were within reasonable limits and all of which would furnish good water. It is of interest to note that the sources investigated by Mr. Hill included most of those which had been reported on at one time or another during the preceding fifty years.

In the winter of 1926, a special water-supply act was passed by the legislature creating the Board of Water Supply of the City of Albany, charged with the duty of selecting a suitable source of water supply and constructing a new water-supply system for the city. This act was nearly identical with the law passed in 1906 creating the Board of Water Supply of the City of New York.

By resolution dated July 8, 1926, the Board of Water Supply of the City of Albany directed the senior author to investigate the feasibility of procuring an adequate water supply of suitable quality from sources lying wholly or mainly within the county of Albany. It seemed to the Board desirable:

1. That the new source of supply should be on the west or Albany side of the Hudson River to avoid carrying the entire supply of the city across or underneath the Hudson River, which is a navigable channel of great commerical importance, recently deepened to 30 ft.
2. That a source of supply should be obtained at suitable elevation,

if possible, so that water could be delivered to all parts of the city by gravity under satisfactory pressure and, furthermore, that the supply, if possible, should be an upland water sufficiently pure so that after adequate storage it could be utilized without filtration. It may be said that at that time there was strong sentiment in the city in favor of continuing the use of the Hudson River as a source of supply, even if it should be necessary to move the intake to a point above Troy and above the mouth of the Mohawk River to avoid the principal sources of pollution. Mr. Hill had investigated this project and found that the cost, including the capitalized cost of pumping and filtration, would range from $12 000 000 to $17 000 000, depending on the point of intake.

3. That the new source of supply should be capable of gradual development and should be sufficient when completed in all its stages to supply the city for at least fifty years in the future.

In his report dated October 11, 1926, the senior author recommended what is known as the Hannacrois-Basic-Catskill gravity supply.* It was also recommended that the first stage of this supply, utilizing Hannacrois and Basic creeks, should be constructed immediately.

The water consumption in the city of Albany in 1926 was about 20 m.g.d., with 45 per cent. of the services metered. Complete meterage was also recommended, and this recommendation is being carried out, so that at the present time the water consumption is considerably below 20 m.g.d.

The Hannacrois-Catskill Project.

The areas included in the project as recommended are shown in Figure 1. They comprise streams lying south and west of Albany in the Helderberg and Catskill Mountain regions and are so located that the dams and most of the conduits will be in Albany County. The project as originally planned contemplates three stages of development, the first stage utilizing Hannacrois Creek with main storage reservoirs at Alcove and Indian fields, about 20 miles southwest of Albany. Basic Creek, a tributary of Catskill Creek, is to be diverted into the Hannacrois drainage basin by means of a tunnel 2 300 ft. in length. These two streams have a total supply area of 48.92 sq. miles and an estimated dependable yield of 30 m.g.d. after supplying water to Ravena, which is located downstream from Alcove on Hannacrois Creek.

The main supply-conduit is to lead from Alcove Reservoir to and across the city of Albany to a new distribution reservoir located on the northerly side of the city of Albany, with flow line at el. 350. This is sufficiently high to supply the entire city under suitable pressure by gravity. This stage of the project, with some modifications hereafter described, is now nearing completion.

The second stage involves the construction of a storage reservoir at the head of Catskill Creek, an intake dam at Preston Hollow, some distance

*Board of Water Supply,— Report on Hannacrois-Catskill Gravity Water Supply for the City of Albany, N. Y.

downstream, and a diversion conduit leading from Preston Hollow to the Hannacrois drainage basin. Partial regulation of the diversion is to be obtained by the Franklinton Reservoir, but the reservoir capacity on Hannacrois Creek will be sufficient to complete the regulation of the flow brought in from Catskill Creek by the second stage of the program. The second stage will give a total drainage area of 93.82 sq. miles, of which, however,

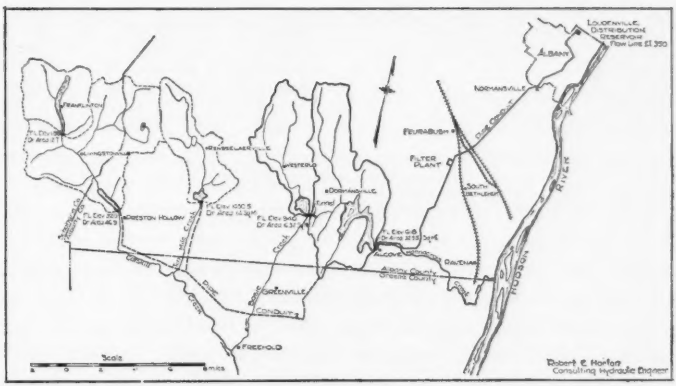

FIG. 1.— GENERAL PLAN OF NEW GRAVITY WATER-SUPPLY SYSTEM.

only 61.62 sq. miles will be completely regulated. This will give a dependable yield of 52.5 m.g.d. or sufficient to supply the city until about 1980.

The third stage of the program contemplates the construction of a storage reservoir of 1 950-m.g. capacity in place of the intake dam at Preston Hollow on Catskill Creek, together with the construction of a diversion dam on Ten Mile Creek, a tributary of the Catskill. This will enable the supply conduit from Catskill Creek to the Hannacrois basin to operate at nearly 100 per cent. load factor and will increase the available supply to 69 m.g.d. or sufficient to supply the city until about the year 2000.

If completed the project will involve the use of 107.82 sq. miles of drainage area. The estimated cost as of 1926 for the first stage is $230 000 per m.g.d. For the increment of 22.5 m.g.d. obtained in the second stage, the estimated cost is $342 000 per m.g.d., the higher price as compared with the first stage resulting in part from the fact that it includes the construction of a second supply conduit from Alcove Reservoir to Albany, bringing the total conduit capacity to a point adequate for the completed project. The estimated cost as of 1926 of the increment of supply of 16.5 m.g.d. added by the third stage is $186 000 per m.g.d.

Taking the present worth of the investment necessary for the construction of the second and third stages of the development, the present value of the total investment is $16 105 395, as of 1926, or $234 000 per m.g.d. The principal features of the project are summarized in Table 1.

TABLE 1.— PRINCIPAL FEATURES OF HANNACROIS-CATSKILL PROJECT.

	Stage.		
	1st.	2nd.	3rd.
Increment of area, sq. mile	48.92	12.7 +32.2†	14.0
Total drainage area, sq. mile	48.92	93.82	107.82
Increment of dependable yield, m.g.d.	30.0	22.5	16.5
Total dependable yield, m.g.d.	30.0	52.5	69.0
Good until	1950	1980	2000
Cost of each stage	$8 409 839	$7 695 566‡	$3 078 537
Cost per m.g.d.	$230 000*	$342 000	$186 000
Increment of storage, bill. gal.	12.00	3.00	1.95
Total storage, bill. gal.	12.00	15.00	17.00

*Not including filter plant or improvements to Albany distribution system.
†Diversion without regulation only from this area.
‡Including a second supply conduit from Alcove to Albany.

It may be noted that this project has been designed not merely with reference to requirements of the immediate future, which will be met by the first stage of the development, but also with reference to securing for Albany an adequate water supply until such time in the future as the city shall have grown to more than three times its present population. While it is perhaps unusual to plan water-supply developments so far into the future, the authors believe that it is in accordance with modern ideas of city planning. Furthermore, in this case there were special reasons why it was desirable to consider not only the work to be carried out at the present time but also future extensions. It was found, for example, that Catskill Creek alone was not a desirable project because of the relatively high cost of securing storage on that stream. Storage was therefore provided for a part of the regulation necessary when Catskill Creek is brought into the system, by increasing the capacity of the storage at present developed on Hannacrois Creek. Thus it became desirable to know at the present time the best method of future utilization of Catskill Creek. The city of New York has had since 1906 permission from the state to utilize Catskill Creek. Before the final adoption of the Hannacrois-Catskill project by the city of Albany, the Board of Water Supply of the city of New York agreed to withdraw its claim of right to use Catskill Creek and to permit the license therefor which had been previously granted by the state to be rescinded.

Abandonment of Hudson River. In adopting the Hannacrois-Basic-Catskill project the Board of Water Supply was, of course, forced to a definite decision on the long mooted question of the abandonment of the Hudson River. Certain important factors entered into this question in addition to those already described:

1. The estimated cost of the first stage of the Hannacrois-Basic gravity supply was $8 500 000, excluding filters, or roundly $9 000 000, including filters, as finally constructed, so that as compared with a new source of

supply from the Hudson River with an intake above the principal sources of pollution and with the cost of pumping continued, as in the past, or perhaps even increased, the economic considerations were strongly in favor of the gravity supply. This fact was a main consideration which led the Board of Water Supply of Albany to the abandonment of the Hudson River and the adoption of a gravity supply.

2. If the present intake in the Hudson River had been retained, an expenditure of some millions of dollars would have been required to provide new mains and distributing-reservoir capacity to meet fire-protection and pressure requirements in the higher-level portions of the city. As will appear later, the new gravity supply meets both these requirements.

The Question of Filtration. Each section of the work as constructed requires approval by the State Water Power and Control Commission. Approval of the first stage of the project now under construction, and which will hereafter be referred to as the Hannacrois-Basic project, was given by the State Water Power and Control Commission in the late fall of 1927. Before describing this project in detail, consideration will be given to one of the principal questions which arose at the hearings before the State Water Power and Control Commission, namely, as to whether or not a filter plant should be constructed in connection with the new supply system.

The original plan contemplated two main reservoirs on Hannacrois Creek; one at Indian Fields, with flow line at el. 645, drainage area of 25.42 sq. miles, storing 7.7 billion gallons, and a second reservoir two miles downstream, with flow line at el. 600, drainage area of 8.45 sq. miles, and 4.4 billion gallons capacity, thus giving a total available storage of 12 billion gallons with minimum draft line at the lower reservoir at el. 575, the ground elevation in the valley at the Alcove Reservoir being about 550. The flow line of the new distribution reservoir at Albany is at el. 350. Water could be delivered by gravity from the Alcove Reservoir with a minimum head of 225 ft. and a maximum head of 250 ft., or it could be delivered directly from Indian Fields Reservoir with a maximum head of 295 ft. It was contemplated that the smaller or Alcove Reservoir would be kept full of pure water drawn from Indian Fields Reservoir after a long period of sedimentation but that if emergency arose, water could be drawn directly from Indian Fields Reservoir, thereby materially increasing the capacity of the supply conduit. It was felt that with two reservoirs, either of which could be used, permitting the other to be held in reserve for additional sedimentation or for algæ treatment, if necessary, a filter plant was not needed, especially in view of the long period of storage, the capacity of the two reservoirs being equal to 400 days' supply with a draft of 30 m.g.d.

The New York State Water Power and Control Commission has, however, authority to require the construction of a filter plant in connection with any new water-supply system. This requirement was imposed on the Albany project. The drainage area tributary both to the Hannacrois Creek and Basic Creek reservoirs is a rugged, upland, semi-mountainous region, underlain by Devonian shales of the Hamilton, Ithaca and Oneonta forma-

tions. There is in general only a shallow mantle of soils of glacial origin of such nature that very little surface material is carried into the streams, and the turbidity of the streams, even in floods, rarely reaches 100 p.p.m. A large part of the drainage area is wild land, much of it sparsely wooded. There are 315 residences on the drainage basin of 48.92 sq. miles, of which 60 are in the village of Westerlo and the remainder are farms. Out of the total number, 65 are vacant or are occupied in the summer only, leaving a total of 250 occupied houses. The resident population on the drainage basin is roundly 1 250, of whom about 300 are in the village of Westerlo. There is practically no summer population within the drainage basin. The village of Westerlo has no water-supply system, is not sewered and with few exceptions the houses are remote from streams and on sandy or gravelly soil. The rural population of about 950 persons is equivalent to 19 inhabitants per sq. mile. Residences adjacent to streams will be provided with sanitary contrivances meeting the requirements of the State Department of Health.

The flowage areas within the reservoir basins are about 40 per cent. wooded, but there are large areas with rock exposed or close to the surface, with only a thin veneer of sandy soil, with sparse vegetation and little organic matter. Muck and swamp areas are flooded only to a negligible extent. Provision was made for drawing water at different levels at the reservoirs.

Under all these circumstances the Board of Water Supply and the authors felt that while filtration might be advantageous for the first few years until the organic matter was removed from the flowage area, filtration could not be considered necessary. Nicholas S. Hill, Jr., in his reconnaissance report did not consider filtration of the Hannacrois or Basic Creek waters necessary. Opinions to the same effect were rendered to the Board of Water Supply by H. K. Barrows, George W. Fuller and Ezra B. Whitman.

The best and practically the only available site for a filter plant was at South Bethlehem, about mid-length of the supply conduit. The interposition of a filter plant on a gravity-supply conduit under these conditions may work serious injury to the hydraulics of a gravity-supply system. This fact does not appear to be generally recognized. To illustrate: Water in the effluent well at the filter plant will be at a practically uniform elevation, 460, giving 110 to 130 ft. available head to deliver water to the distribution reservoir at Albany at flow line and minimum stages of the reservoir, respectively. Alcove Reservoir as finally constructed has a flow line at el. 625 and minimum draft line at el. 575. It would be necessary to design the conduit of sufficient capacity to deliver the maximum demand of 45 m.g.d. to the filter plant with Alcove Reservoir at el. 575. The additional head of about 50 ft. available when Alcove Reservoir is full cannot be used to increase the delivery to the city of Albany without by-passing the filter plant, whereas if the filter plant was omitted, not only could this increased head be utilized but additional head, obtainable from the Indian Fields Reservoir, could be used as originally proposed. The net result is that dividing the

conduit line into two sections by a filter plant with a constant level fixed at the filter plant cuts down the maximum capacity of the conduit by about 4 m.g.d.

Since, in view of the requirement for a filter plant, advantage could not be taken of increased head at Indian Fields, and since, furthermore, the advantage of two reservoirs was diminished, it was decided to omit the Indian Fields Reservoir and raise the flow line at Alcove from el. 600 to el. 625, giving practically the same storage as before, or 12 billion gallons net.

Some saving was effected by the substitution of a single reservoir at Alcove in place of the combined Alcove and Indian Fields reservoirs, so that with other economies it has been possible to construct the Hannacrois-Basic supply system at a cost to completion of something less than $9 000 000 including approximately $1 000 000 for the filter plant, as compared with an estimated cost without filter plant of $8 400 000.

Yield of Supply Areas. At the time the original report was made by the senior author in 1926 there were available records of flow of Hannacrois Creek for portions of two years. No records were available on Basic Creek. There was a fairly good record of flow of Catskill Creek for 25 years, and fortunately there was an excellent record of rainfall at Albany over 100 years. The Board of Water Supply felt that Hannacrois-Basic creeks, if adopted should have a dependable yield of at least 30 m.g.d. or 50 per cent. in excess of the 1926 demand. The minimum yield of Hannacrois Creek is but a few hundred thousand gallons per day. Most of the citizens of Albany had seen this stream, if at all, only in the low-water season. There were many who averred that the reservoirs would never fill and that the supply was grossly inadequate for the city of Albany. As usually happens in such cases, the less they knew about it the more freely some of them expressed their opinions. Nicholas S. Hill, Jr., had estimated the available yield of Hannacrois-Basic creeks at 35 m.g.d. The data available to him were, however, meager.

Basic Creek lies between Hannacrois and Catskill creeks. The ratio of yield of Hannacrois Creek to that of Catskill Creek was determined for the period of actual gagings of the former, and this ratio was applied to the 25-year record of Catskill Creek for the determination of the yield of Hannacrois-Basic creeks. A correlation was then obtained between the yield of Catskill Creek and the Albany rainfall record and was used to determine the probable yield of Catskill Creek for a period of 100 years. The object of this study was to find out whether periods of drought of greater severity than any which had occurred during the 25 years covered by Catskill Creek gaging records were to be expected. So far as could be determined by this method it was found that the severe drought periods which occurred during the years 1906–1920 in eastern New York had not been exceeded in intensity or duration at any time within the past 100 years. Using this as a basis, a study was made by mass diagram and storage-

requirement curve methods which indicated that a dependable yield of 30 m.g.d. could be obtained from Hannacrois-Basic creeks with 9.85 billion gallons' storage. The storage was increased to 12 billion gallons (1) to provide for partial regulation of Catskill Creek water to be brought into the main reservoir in connection with the second stage of the project; and (2) as a factor of safety. It may be remarked in passing that there is some uncertainty in all hydrologic calculations and estimates of stream yield.

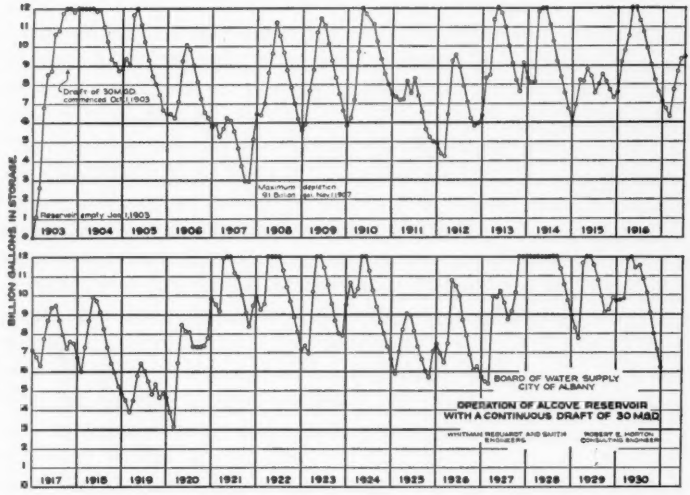
BILLION GALLONS IN STORAGE
Draft of 30 M.G.D. commences Oct. 1, 1903
Reservoir empty Jan. 1, 1903
Maximum depletion 9.1 Billion gal. Nov. 1, 1907
1903 1904 1905 1906 1907 1908 1909 1910 1911 1912 1913 1914 1915 1916
1917 1918 1919 1920 1921 1922 1923 1924 1925 1926 1927 1928 1929 1930
BOARD OF WATER SUPPLY
CITY OF ALBANY
OPERATION OF ALCOVE RESERVOIR
WITH A CONTINUOUS DRAFT OF 30 M.G.D.
WHITMAN REQUARDT AND SMITH ENGINEERS
ROBERT E. HORTON CONSULTING ENGINEER

FIG. 2.— OPERATION OF ALCOVE RESERVOIR WITH A CONTINUOUS DRAFT OF 30 M.G.D.

The simplest and perhaps the most satisfactory method of including a factor of safety in estimates of dependable yield of water-supply sources is by providing storage in excess of the apparent amount required, as was done here.

The estimated yield was checked by computing month by month the operation of the system with 30-m.g.d. draft and 12 billion gallons' storage, beginning with the reservoir empty on January 1, 1903. The results are shown in Figure 2. Storage-requirement determinations based on the mass diagrams, while simple and extremely useful, have their limitations. It is satisfying to the engineer and more convincing to the layman to follow up a mass-diagram study by an operation diagram such as this. Figure 2 shows that the reservoir would fill completely and waste would occur only in the wetter years. The maximum depletion during the period covered by the diagram was 9.1 billion gallons, on November 1, 1907. George W. Fuller and Professor H. K. Barrows checked this estimate of available yield by independent methods and studies and confirmed the conclusion that a

net dependable supply of at least 30 m.g.d. and probably somewhat more could be derived from Hannacrois-Basic creeks. These studies were made in 1926 and 1928.

The rainfall of the summer and fall of 1929 was deficient. This was followed by a winter of meager snowfall and no replenishment of groundwater. The soil in the drainage basin at the end of the winter of 1929–1930, after the snow had melted, was so dry that the pipe-line trench excavator stirred up clouds of dust in February in digging below an 18-in. layer of frost. Again, the summer of 1930 has been abnormally dry. This has afforded an opportunity to check the preliminary estimates of yield, although the dry season has not yet terminated. As shown in Figure 2, on October 31, 1930 there would have remained 6.3 billion gallons of water in storage.

In the JOURNAL OF THE NEW ENGLAND WATER WORKS ASSOCIATION for March, 1924, one of the authors estimated the minimum precipitation to be expected at Albany, N. Y., as 68 per cent. of the mean rainfall.* This was based on an 88-year record from 1826 to 1913, for which period the mean rainfall was 37.98 in., giving a predicted minimum annual rainfall of 25.83 in. The lowest previously recorded rainfall was 69 per cent. of this mean in 1913. The U. S. Weather Bureau record at Albany for the calendar year 1930 was 25.52 in. This is the lowest value on record. The mean rainfall for the period 1826 to 1930, inclusive, is 37.18 in. The predicted minimum annual rainfall based on this mean is 25.28 in. or a little less than the 1930 rainfall. In using the ratio method for determining the predicted minimum rainfall, the longest and most reliable basis of determining the mean rainfall should be utilized.

The lowest rainfall amounts at Albany in different years have been as follows:

	Inches.
Predicted minimum	25.28
1930	25.52
1913	26.39
1896	27.88
1864	27.94
1909	27.96
1908	28.41
1910	28.51

The rainfall in 1929 was 31.68 in., and this was preceded by an excessively wet period. Unless the drought conditions of 1930 should be repeated during the next two years, the available yield of streams for water-supply purposes will not fall so low as in the period of three dry years 1906–1908, inclusive. The estimated capacity of the new Hannacrois-Basic creek gravity supply was based on the record for those years.

*Horton, Robert E. Determining the Mean Precipitation on a Drainage Basin. JOUR. N.E.W.W.A., March, 1924, p. 22.

The measured average daily yield of the supply sources, aggregating 48.9 sq. miles, in different years has been as follows:

	m.g.d.
1927	64.87
1928	44.64
1929	52.22
1930 (to November 1)	26.10

To maintain a supply of 30 m.g.d. in dry years, like 1930, requires the use of storage carried forward from preceding years.

Design and Construction.

The Program. The general scheme of a new gravity water supply for the city of Albany has been previously described as consisting of three stages of successive development. The work which is now under construction and rapidly nearing completion embraces the first stage which utilizes the flow of Hannacrois Creek and Basic Creek. The principal construction features of this first stage include:

1. A dam on Basic Creek forming a reservoir of 670-m.g. capacity. This dam functions principally for the purpose of diverting the flow of Basic Creek through a tunnel into the Hannacrois Creek drainage basin, and in addition the reservoir behind the dam very effectively regulates the flood flows of Basic Creek in excess of the tunnel capacity of 300 c.f.s. This dam was completed in 1929.

2. The Basic Creek diversion tunnel for carrying the flow of Basic Creek into the Hannacrois Creek drainage basin. This tunnel is 5 ft. wide by 6 ft. high by 2 810 ft. long at a maximum depth of 90 ft. This tunnel was completed in 1929.

3. The Alcove Dam forms an impounding reservoir on Hannacrois Creek with a usable capacity of 12 billion gallons. This dam was completed in September, 1930, although the outlet gates were closed in December, 1929. The Alcove Reservoir is now filling and contained on November 1, 1930 an available supply amounting to 7 billion gallons, being equivalent to one year's supply at the present rate of water consumption. Use of this water is delayed pending the completion of the filter plant and distribution reservoir.*

4. A 48-in. cast-iron supply conduit, having a total length of 105 000 ft., leading from Alcove Dam to Loudonville Distribution Reservoir. This conduit is now completed.

5. A rapid sand-filter plant midway between Alcove Reservoir and the city of Albany, having a normal capacity of 32 m.g.d. This filter plant will be completed during the winter of 1930–1931.

6. Loudonville Distribution Reservoir, having a gross capacity of 100 m.g. This reservoir is located in the northerly part of the city of Albany, in a natural depression on the high land adjoining the west bank of the Hudson River. The new source of supply enters the city from the south. Construction of this reservoir is nearing completion.

7. Rearrangement of water distribution-system, including connections to the new 48-in. supply conduit, revision of the use of the existing distri-

* Reservoir filled April, 1931.

bution reservoirs and installation of the pressure regulating valves to supply water to the low-service district from the high-service feeder mains.

In addition to the special items of work outlined above, there are various minor contracts including the clearing and grubbing of the reservoir areas, removal of cemeteries, relocation of highways, special construction at railroad and stream crossings and sanitation of watersheds.

At the time of the preparation of the original report to the Board of Water Supply it was estimated that the first stage of the project could be completed within three years after definite approval by the State Water Power and Control Commission when the work could be actively prosecuted. This prediction was on the basis of our original recommended plan, the estimated cost of which, not including the construction of the filter plant, was $8 400 000. The Board of Water Supply was further advised that the inclusion of a filter plant would increase this cost to $9 400 000, and would affect the time of completion of the project.

The approval by the State Water Power and Control Commission was granted in October, 1927, and after other minor legal restrictions were removed, the Board of Water Supply was in a position to authorize the commencement of the design and construction in January, 1928, through the appointment of Whitman, Requardt and Smith, Engineers, with Robert E. Horton, Consulting Engineer. The construction of a filter plant was included in the requirements of the State Water Power and Control Commission and in spite of this increased cost, amounting to approximately $1 000 000, the actual cost of this first stage of the development, owing to savings in other items, will be approximately $9 000 000. At the present writing the construction work on the entire project is over 90 per cent. complete, and it is believed that the entire work will be finished by June 1, 1931.

The total cost of all physical construction will be about $7 200 000. To this figure must be added the cost of acquisition of lands, purchase of riparian rights, and such overhead charges as engineering, legal and administration fees, together with interest during construction. These latter charges increase the total to about $9 000 000.

It is believed that the carrying out to completion of a project of this size within a total period of slightly more than three years is very unusual, and it is interesting to review the progress of the work during this period.

Figure 3 illustrates graphically the rates of progress of performing the engineering design, the carrying out of construction, and acquiring lands and riparian rights. It is important in a project of this character to determine at the beginning the sequence in which the various contracts must be started in order that the entire work may be completed within a minimum length of time and at the same time coördinate with the placing of the various units in operation. This is particularly important in a situation such as that at Albany where the existing supply is subject to high operating

costs for pumping and an early abandonment of the existing source is of considerable financial benefit to the city. Figure 3 shows that it was necessary to complete 60 per cent. of the engineering design within the first year and practically all the engineering design within the first two years. While

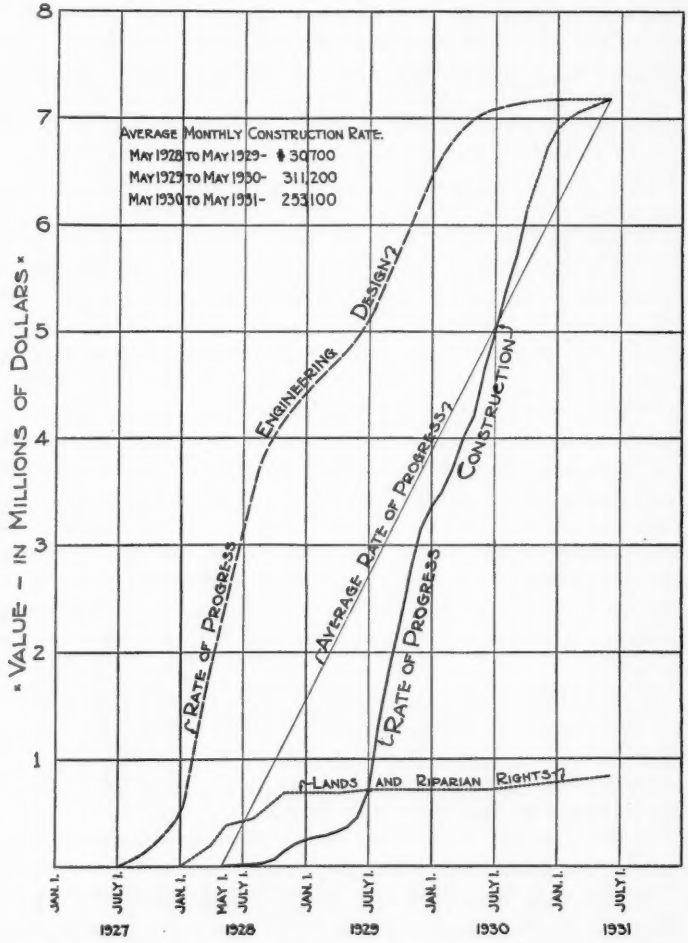

FIG. 3.— RATES OF PROGRESS, ENGINEERING DESIGN, CONSTRUCTION, AND ACQUISITION OF LANDS AND RIPARIAN RIGHTS.

the engineering design was in progress, it was also necessary to make surveys immediately for the acquisition of land so that these lands could be occupied as soon as the commencement of construction required. This resulted in the completion of all land surveys within the first ten months.

By starting the design in January, 1928, it was possible to place the first contract under construction in May of the same year. The diagram indicates that from May, 1928 to May, 1929, the construction rate was \$30 700 per month; from May, 1929 to May, 1930, \$311 200 per month; from May, 1930 to May, 1931, the estimated monthly construction rate is \$253 100 per month. Figure 4 shows graphically the construction period and the value of each of the principal construction contracts.

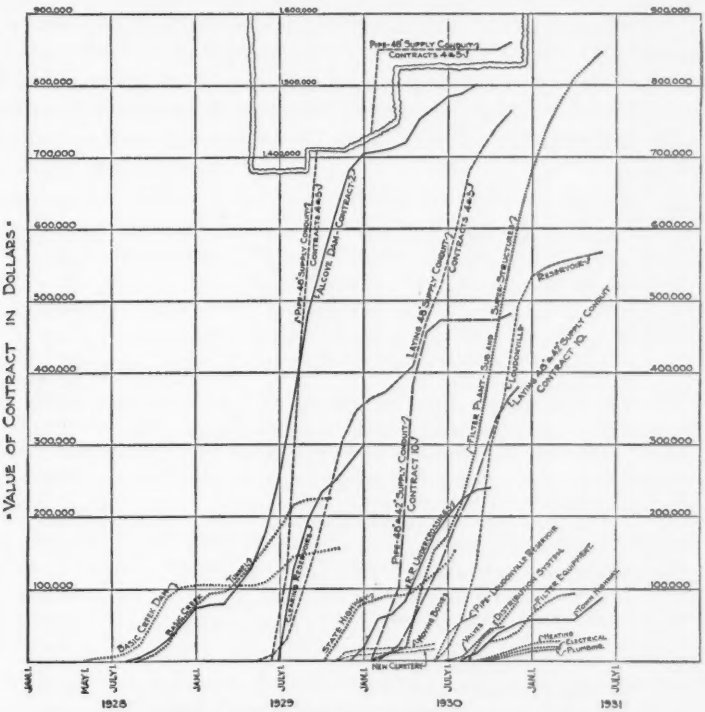

FIG. 4.— RATES OF PROGRESS OF PRINCIPAL CONSTRUCTION CONTRACTS.

In respect to the principal construction contracts, it was desirable to complete Basic Creek Dam and Basic Creek Tunnel at an early date in order that the water of Basic Creek could be diverted into the main storage reservoir on Hannacrois Creek. The contract of next importance was that of Alcove Dam, which was started at an early date in order that it would be sufficiently completed to commence the impounding of the water in Alcove Reservoir on January 1, 1930. Incidental to the storage of this water, it was necessary to complete the clearing of the reservoirs by this same date and to construct a new cemetery and to move existing cemeteries before these reservoir areas were flooded.

With these key contracts under way the succeeding major contracts, including two sections of the 48-in. supply conduit and the filter plant, were then placed under construction. Then followed the construction of Loudonville Distribution Reservoir, after which the remaining smaller contracts were started.

This order of sequence of contracts has proved to be very satisfactory in coördinating the completion of the entire work. It would have been of advantage to start the filter plant at a slightly earlier date. However, a large amount of engineering design is required on work of this character, with the result that it would be extremely difficult to complete satisfactorily filter-plant contract drawings of sufficient detail within a shorter period. The following is a summary of the principal construction contracts of the new Albany Water Supply giving the approximate total cost of each contract:

Contract No.	Item.	Approximate Cost.
1	Basic Creek Dam.	$160 000
2	Alcove Dam.	800 000
3	Basic Creek Tunnel.	248 000
4 and 5	48-in. supply conduit; furnishing pipe.	1 460 000
4 and 5	48-in. supply conduit; laying pipe.	765 000
6	Clearing and grubbing Alcove and Basic reservoirs.	300 000
7	Railroad undercrossings.	241 000
9	Bituminous Macadam state highway.	186 000
10	48-in. and 42-in. supply conduit; furnishing pipe.	482 000
10	48-in. and 42-in. supply conduit; laying pipe.	395 000
11	Removal of bodies from cemeteries.	25 000
12	Construction of new cemetery.	16 000
8	Filter plant; sub- and superstructures.	850 000
13	Filter plant; valves.	48 000
14	Filter plant; plumbing.	17 000
15	Filter plant; heating.	28 000
16	Filter plant; electrical work.	19 000
17	Filter plant; pumps.	4 000
18	Filter plant; equipment.	94 000
19	Loudonville Reservoir.	632 000
20	Distribution-system improvements, including connections.	160 000
21	Bituminous Macadam town road.	88 000
22	Waterbound Macadam town road.	14 000
	Miscellaneous work performed under special agreement, including supporting of tracks by the railroads.	150 000
	Total	$7 182 000

The remainder of this paper will briefly describe the general features of each of these contracts including an outline of the nature of the work and unit prices on the principal items. On some of these contracts there are certain features of design, a description of which it is believed will prove of interest. In other instances certain methods of construction were used, the

results of which it is believed will prove of value in planning other work of similar nature.

Basic Creek Dam. This structure consists of an earth dam of compacted embankment with a central concrete core wall. It is located at a point on Basic Creek where the drainage area is 16.17 sq. miles, and it was built to such a height as to impound 670 m.g. of water. This dam was constructed principally for diversion purposes in order to divert the flow of Basic Creek through Basic Creek Tunnel into the Hannacrois Creek watershed. The capacity of Basic Creek Tunnel is approximately 300 c.f.s., sufficient to carry the ordinary flood flows of this stream. The storage of 670 m.g. behind Basic Creek Dam was easily provided by constructing this dam to a height of about 25 ft., thus making it possible to store and utilize practically all the extreme flood flows of Basic Creek, resulting in complete regulation of this stream.

The total length of the dam including the concrete spillway is 872 ft. The embankment section has a top width of 15 ft., an upstream slope of 1 on 3 and a downstream slope of 1 on $2\frac{1}{2}$. The upstream slope is covered with stone paving 18 in. in thickness. The center concrete core wall is 24 in. thick at the top with a batter on each side of 1 in. in 1 ft. Each face of the concrete core wall is reinforced with $\frac{3}{4}$-in. round rods spaced on 12-in. centers in both directions.

Work on this contract was started in May, 1928, and it was originally contemplated that this dam should be completed during the construction season of that year. The summer of 1928, however, offered very unfavorable working conditions, with frequent rains, which resulted in flooding the flat valley in which the dam was constructed. Due to the frequency of rains and the soft condition of the bottom land the contractor was unable to use teams or ordinary trucks for the hauling of materials and the placing of earth embankment. It became necessary for him to use trucks and tractors equipped with caterpillar treads.

A concrete mixer of the paving type was used in placing all the concrete in both the core wall and the spillway section of the dam. The concrete aggregates, including sand and stone, were stored alongside of the batching plant. The proper proportions of the sand and stone, together with the required bags of cement, were loaded on trucks at this plant and hauled to the paving mixer. The mixer was moved immediately alongside of the section to be poured. Concrete below the ground, such as the cut-off wall, was poured directly from the mixer. The concrete above the level of the ground was deposited from the mixer into a bottom-dump bucket which was lifted by a crane and then deposited into the top of the form. The core wall sections which were about 20 ft. high were all poured in one lift and in lengths of from 30 to 50 ft.

The unit prices of the principal items of the contracts were as follows:

Consolidated embankment	36 000 cu. yd.	@	$.65
Concrete masonry in spillway	5 000 cu. yd.	@	11.00
Concrete masonry in core wall	1 400 cu. yd.	@	12.50

Alcove Dam. Alcove Dam forms the main impounding reservoir of the new water supply. This dam is constructed across Hannacrois Creek at a point where the drainage area is about 32.55 sq. miles. At this point the stream bottom is at el. 544, the spillway at el. 618 and the top of the earth embankment at el. 625. The minimum draft line in the reservoir behind this dam is assumed at el. 575. The available storage in Alcove Reservoir is 12 billion gallons. The over-all length of the dam is 2 177 ft. and the maximum height above stream bottom, 81 ft.

This dam is formed of compacted earth embankment with a center concrete core wall. The top width of the embankment is 15 ft. with an upstream slope of 1 on 3 and a downstream slope of 1 on $2\frac{1}{2}$. Midway along the downstream slope is a berm 20 ft. in width. The upstream slope is covered with stone paving 18 in. in thickness down to el. 575. Below this elevation the slope is protected with stone riprap 36 in. in thickness.

At the extreme south end of the dam on the downstream side there is located a minor depression which joins Hannacrois Creek approximately 1 000 ft. below the dam. This offered a very favorable location for the discharge of the flood waters from the reservoir, and the concrete overflow spillway 300 ft. in length was constructed at this location. This resulted in considerable economy in the construction of the dam for the following reasons: (1) rock foundation existed only a few feet below the surface of the ground; (2) required height of concrete overflow section was only 18 ft.; (3) the natural depression was of sufficient cross-sectional area to provide for a discharge of 15 000 c.f.s. from the reservoir with a very small amount of grading and paving construction; (4) the flood waters were returned to the stream a safe distance below the dam.

Construction work on this contract was started in August, 1928. It was realized that a contract of this size, amounting to approximately $800 000, would require at least eighteen months to complete, and the principal item of the work, the placing of 340 000 cu. yd. of compacted embankment had to be coördinated with the remaining work of the contract together with making proper provision for taking care of stream flows during the placing of this embankment. The specifications provided that during the winter and spring of 1928–1929 the contractor should proceed with all items of the construction work on both sides of the valley including the stripping of the site, the excavation for the cut-off wall, the building of the concrete cut-off and core walls, and the construction of the gatehouse and intake. He was, however, forbidden to place earth embankment or core wall in the bottom of the valley until the period of low flow during the summer of 1929. This method of procedure was followed out.

In order to carry the flow of Hannacrois Creek through the dam, a reservoir drain was constructed along the stream bank during the winter and spring of 1929. This drain is 8 ft. wide and 6 ft. high, and has a capacity of 1 260 c.f.s. with water surface behind the dam at el. 575 and a capacity of 1 400 c.f.s. with water surface at el. 580. The specifications

directed that the contractor should divert Hannacrois Creek through this drain during the months of May, June and July, at which time he should build up the embankment as rapidly as possible to at least el. 580, at which point the drain would have a capacity of 1 400 c.f.s. The placing of the embankment was started the latter part of May. By June 30, it had reached el. 575 and by July 31, it had reached el. 585. During this period the reser-

FIG. 5.— RESERVOIR DRAIN AT ALCOVE DAM.

voir drain was amply able to carry the flow of the stream. The maximum discharges of Hannacrois Creek during this summer period of 1929 were as follows:

Month.	c.f.s.
April	1 193
May	234
June	61
July	17
August	13
September	19
October	203

In order to develop the maximum effective discharge through the reservoir drain a unique design was employed for the outlet of this drain. Figure 5 is a photograph showing the completed reservoir drain before the earth embankment had been placed. This drain, with a cross-sectional area of about 40 sq. ft., provided for a velocity of flow of something over 30 ft. per second with a discharge of water behind the dam at about el. 575. If this cross-sectional area had been carried through for the full length of the drain, the velocity head lost at the outlet would have amounted to approximately 14 ft. In order to conserve most of this velocity head, the cross section of the drain at the outlet was enlarged to an area of 96 sq. ft. so that

for a discharge of 1 200 c.f.s. the velocity head saved amounted to about 11½ ft. This meant that for a fixed flow in Hannacrois Creek the embankment at the dam could be 11½ ft. lower with the same factor of safety; or inversely, for a fixed height of embankment the capacity of the reservoir drain was considerably increased.

The earth embankment was obtained from borrow pits located on the hillside immediately above the dam. The average haul of this material was about 500 ft. Practically all this embankment was placed during the period May 15 to November 15. During the months of June, July and August, the contractor operated on two shifts of ten hours each; during the remainder of the time only one shift of ten hours was used. Table 2 gives the results of placing this embankment, together with the equipment used.

TABLE 2.— CONSTRUCTION OF ALCOVE DAM — CONSOLIDATED EMBANKMENT.

Month, 1929.	Cu. Yd.	Shifts.	Cu. Yd. per Shift.	Shovels.	Teams.	Trucks.	Spreaders.	Rollers.	Haul, Ft.
May	10 001	18	556	3	10	7	2	1	300
June	46 937	53½	878	4	16	6	2	2	400
July	94 492	58	1 628	5	18	6	2	2	600
August	75 677	48	1 578	5	18	13	2	2	700
September	47 273	22	2 143	5	18	13	2	2	500
October	41 185	23¼	1 769	5	18	13	2	2	800
November	8 410	16	527	5	18	13	2	2	800

By November 30, 1929, the main embankment of the dam had been carried up to the full height, or el. 625. At the same time the gatehouse, intake structure and outlet conduits were completed. There remained only the construction of the upper portion of the concrete spillway at the extreme south end of the dam. It was thus possible for the city to close the gate in the reservoir drain on December 13, 1929, since which time this reservoir has been impounding water preparatory to the introduction of the supply to the city during the early part of 1931. About April 1, 1930, the contractor was able to resume work on this contract, and by June this dam was entirely completed.

The unit prices on the principal items of the contract were as follows:

Item.	Cu. yd.	Unit price.
Stripping excavation	24 000	$1.00
General earth excavation	22 000	.60
General rock excavation	13 000	1.30
Rock excavation in trench	6 500	2.80
Consolidated embankment	340 000	.60
Slope paving	11 000	4.00
Concrete masonry in spillway and cut-off walls	18 000	9.50
Concrete masonry in core walls	10 000	14.00

Basic Creek Tunnel. This tunnel was constructed for the purpose of carrying the flow of Basic Creek through the dividing ridge into the Hannacrois Creek drainage basin. The main tunnel section is 2 810 ft. long at a maximum depth of 90 ft. below the ground surface. A cut-and-cover concrete conduit carries the water from Basic Creek to the tunnel entrance, and

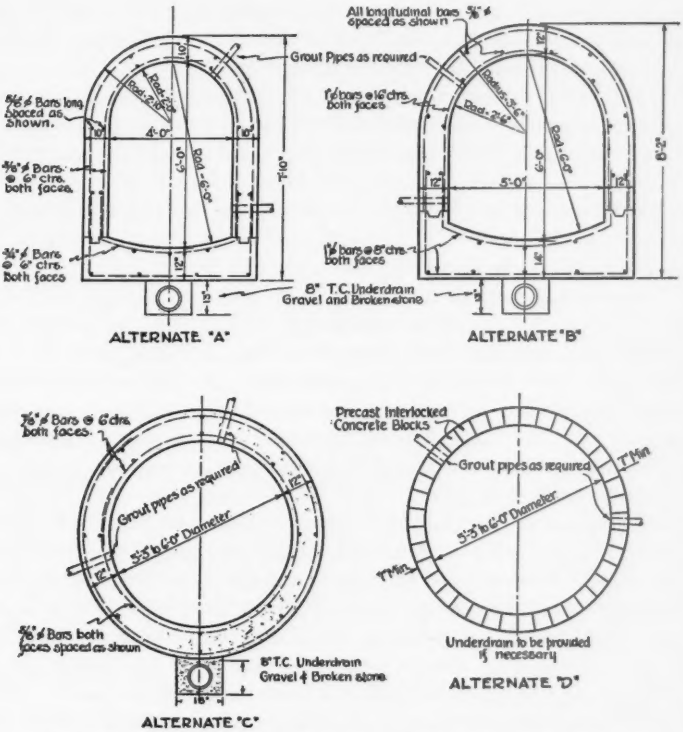
Grout Pipes as required
5/8" ϕ Bars long spaced as shown.
3/8" ϕ Bars @ 6" ctrs. both faces.
3/8" ϕ Bars @ 6" ctrs. both faces
8" T.C. Underdrain Gravel and Broken stone
ALTERNATE "A"
All longitudinal bars 5/8" ϕ spaced as shown
1" bars @ 12" ctrs. both faces
1" bars @ 8" ctrs. both faces
ALTERNATE "B"
3/8" ϕ Bars @ 6" ctrs. both faces
Grout pipes as required
5'-3" to 6'-0" Diameter
5/8" ϕ Bars both faces spaced as shown
8" T.C. Underdrain Gravel & Broken stone
ALTERNATE "C"
Precast Interlocked Concrete Blocks
Grout pipes as required
5'-3" to 6'-0" Diameter
Underdrain to be provided if necessary
ALTERNATE "D"

FIG. 6.— ALTERNATE CROSS SECTIONS OF BASIC CREEK TUNNEL.

a similar conduit conveys the water from the tunnel outlet to the tributary of Hannacrois Creek into which it discharges.

It was desired to build a tunnel of a capacity of not less than 200 c.f.s. This would have required an inside diameter of 63 in., and it was believed that a structure of this small inside dimension would probably prove uneconomical to build. Before preparing the plans a number of contractors experienced on tunnel construction were consulted, and there was considerable diversity of opinion as to the economical size and type. Preliminary test borings showed that both earth and rock would be encountered in driving the tunnel. It was decided to prepare plans calling for four alternate types of structure. These four types are shown in Figure 6. Section A

provides for a concrete lining 12 in. in thickness with inside dimensions of 4 ft. in width by 6 ft. in height. This section has an arched roof with vertical side walls and a curved invert. Section B is similar to Section A except that the width is 5 ft. instead of 4 ft. Section C provides for a circular outline from 5 ft. 3 in. to 6 ft. in diameter with a lining of concrete 12 in. in thickness which was to be poured in place. Section D is a circular outline of the same dimensions as Section C. This type of tunnel was to be constructed of hexagonal precast concrete blocks which would be designed to be interlocking and to be placed by means of a tunnel shield.

Bids for this contract were received on the basis of a unit price per foot of completed tunnel through various types of excavation, including earth, rock and earth, and rock. The lowest bid on this contract was submitted by S. J. Groves and Sons, Minneapolis, Minn., and was based on the construction of Section B. The next lowest bid was $25 000 higher and was based on the construction of Section D. Construction work on this contract was started in August, 1928 and completed in November, 1929.

The so-called earth section of the tunnel occurred at each end and was composed of a stony, glacial till, which might be termed a silt hardpan. Most of this material was dry and hard to excavate. The contractor first tried to use air spades, which proved unsuccessful; he then used drilling and blasting methods for breaking up the material. The tunnel was driven from each end, and the excavated materials were removed by means of electric mining-locomotives and steel dump-cars operating on a narrow-gage track. Ventilation was provided by exhaust fans at each end of the tunnel connected to a steel pipe-duct that was extended as excavation progressed.

After the hardpan was excavated, steel tunnel liner-plates were erected to the outside dimensions of the tunnel and the space behind these plates was packed with excavated materials and later grouted. These steel liner-plates, which were 16 in. by 36 in. by 3/32 in. thick proved adequate for shoring the excavation where the material was dry. At certain locations near the inlet of the tunnel where the earth overburden was about 20 ft. and where ground-water conditions existed, the pressures on these liner-plates were too great and it was necessary to resort to timbering. The failure of the plates in this instance was probably due to the fact that they were not of sufficient thickness and were not properly stiffened by ribs or corrugations.

The tunnel was completely excavated before any of the concrete lining was placed. The first portion of the concrete lining that was placed consisted of the floor, which was constructed for the full length of the tunnel. The industrial track was then laid on top of the concrete floor and Blaw-Knox steel forms were used for sidewalls and roof. Due to the limited headroom the contractor decided that it would be uneconomical to use a pneumatic machine for the placing of concrete. All concrete was placed by hand. It was mixed at the end of the tunnel and transported by steel dump-cars on the industrial track. Steel plates were placed on top of the concrete

floor, and the concrete was dumped from the cars to this steel platform. It was then placed by hand into the forms. The straight sidewall-forms were first erected and were filled with concrete up to the springing line. The forms for the arch were then placed with the exception of a key plate in the center of the top. Concrete was then placed in the forms up to the level of the key plate. After it had reached this level, the key plates were placed in position, one at a time, and the concrete was then rammed back behind the forms. It was found that jarring of the steel forms by means of a wooden maul proved very effective as a substitute for spading and for the elimination of honeycomb spots. After the contractor had organized his work properly, he was able to place about 600 lin. ft. of floor per week and a maximum of 350 lin. ft. of sidewalls and crown.

The specifications provided for the placing of grout behind the tunnel lining after it was completed. For this purpose grout pipes were placed in the lining about 30 ft. apart along the center line of the crown and along each side. Each batch of grout was composed of $\frac{1}{3}$ bag of cement, 1 cu. ft. of fine sand and sufficient water to make a total volume of 3 cu. ft. It was applied under a pressure of about 50 lb. per sq. in.

It was originally assumed that 200 cu. yd. of grout would be required for the full length of 2 810 ft. This character of work is very difficult to estimate, and it actually required 1 600 cu. yd. to completely grout behind the lining of the tunnel.

The unit prices applying to the principal items of the contract were as follows:

Item.	Price per cu. yd.
Concrete masonry in cut-and-cover conduit	$20.00
Completed tunnel in earth	50.00
Completed tunnel in earth and rock	55.00
Completed tunnel in rock	60.00

The total length of the tunnel including the inlet and outlet conduits, is 3 736 ft. This contract was completed at a total cost of approximately $248 000.

Supply Conduit and Railroad Undercrossings. The main supply conduit has a length from Alcove Reservoir to Loudonville Distribution Reservoir of 105 000 ft. Starting at Alcove Reservoir, there were alternative locations for the first 12 600 ft. (1) in the Hannacrois Creek Valley, with seven stream crossings; or (2) along the state highway, mainly through the front yards of residences.

Hannacrois Creek has a boulder-filled valley, and it was expected that trench excavation in this valley would be tedious and involve considerable hand work. Nevertheless, this would avoid crossing the front yards of a large number of residences, with consequent annoyance and delays in securing property rights. Hence, the valley route was adopted. The remaining 31 400 ft. of the conduit line leading to the filter plant is through

rough, hilly country. The trench through much of the distance is in rugged, fissured limestone rock, overlain with shallow earth.

The filter plant is on the crest of the Helderberg escarpment. From the foot of this escarpment to Loudonville Reservoir, a distance of about 61 000 ft., the pipe line is wholly in earth, the soil consisting generally of a fine silt resembling loess in its characteristics which when reasonably dry will stand vertically in the sides of a trench. This soil extends to a depth of about 8 ft. and is underlain by a fat blue clay. The surface is generally plane, sloping gently except where traversed by deep ravines.

Compared, for example, with highway and railway location, the engineering economics of water-supply conduit-locations seem to have received little attention. In the authors' experience it is impossible in traversing rough country to make the best possible location of a supply conduit without a preliminary survey. In the present instance a preliminary survey was made with azimuth by Polaris and checked by solar observations. The location of the base line was determined by preliminary reconnaissance, using U. S. Geological Survey maps, with the idea of running the line as nearly straight as possible from Alcove to Albany. Topography was taken to distances of 50 to 300 ft. on either side of the base line and fences, houses and other landmarks were accurately located. Alternate routes were surveyed for some portions of the line. Using the maps obtained from these surveys it was found to be easy to make not only a satisfactory but undoubtedly the best possible location of the supply conduit, even in rough country. A final survey of the adopted location was then made.

Conditions with reference to the economic location of water-supply conduits have changed materially within the past few years. Not so long ago the location of a conduit, so that as great a portion as possible of the excavation could be made by machine, involved running around steeper hills and knolls. In the present instance, all the excavation, including rock after blasting, and all the backfill, was done by machine, even though on the steeper slopes the trench machines had to be held in position on a platform by wire rope cables anchored at the top of the hill.

Under these conditions a location which is a straight line in plan was nearly attainable. Nevertheless, a location consisting, in plan, of a straight line, may involve an undue amount of vertical curvature, with consequent special bends and with the necessity of an increased number of air valves at summits and blow-offs in depressions as compared with a line curved in plan but with less vertical curvature. Not only is the increased cost of special bends and the increased cost and inconvenience in operation due to an excess of air valves and blow-offs, to be considered, but where a trenching machine is used both for excavation and for placing the pipe in position, as was done in this instance, every special bend in the line involves an appreciable loss of time as compared with the use of straight pipe. In the present instances minor knolls were smoothed off and steeper slopes flattened by preliminary cuts with the excavator. In this way the amount of vertical

curvature in the line was considerably reduced and the use of tangent lines made possible where horizontal deflections around hills would otherwise have been necessary. This method also permitted the substitution of joint deflections in place of vertical bends in a number of instances.

At the time of adoption of the project it was realized that a number of types of material were available for supply conduit, including cast iron, riveted steel, lock-bar steel, ingot iron, welded steel and reinforced-concrete pipe with steel diaphragm. Some of these types of conduit had only recently come into use, and there was little experience available from which to judge their permanence and utility. Neither the Board of Water Supply of Albany nor its engineers felt it to be desirable to permit free-for-all competitive bids from manufacturers of different types of conduits, as has been done in some instances. On the other hand, manufacturers of all the different types of conduit mentioned were asked to present to the engineers confidential data, including approximate cost estimates, setting forth the characteristics and alleged advantages of the particular material which they manufactured. All these data were summarized in a report in such form as to permit ready comparison of the different types of conduit available, with reference to (1) durability; (2) first cost; (3) sustained cost based on a reasonable estimate of useful life; (4) freedom from breaks, leakage or interruptions in operation; and (5) speed and facility of construction under the existing conditions.

One of the governing features of the situation and one which was given much weight in the final decision of the Board of Water Supply to adopt cast-iron pipe was the fact that the reserve supply in the distribution reservoir at Albany would only be sufficient for a period of three days with a consumption rate of 30 m.g.d. and that for a period of 25 or 30 years, at least, there would be only a single supply conduit leading from Alcove Reservoir to the city. It was therefore desirable to select a type of conduit material which should be unquestionably dependable for continuous service day in and day out throughout this period of time. It was fully realized that breaks would occur even in the best built cast-iron supply conduit when new, but the experience of the authors with a number of long gravity supply conduits of cast iron through rough country had shown that such breaks generally result from either (1) bell-hung pipes, which are not well supported by backfill underneath; (2) defective pipes; or (3) inadequate anchorage at bends. Breaks from these causes as a rule occur, if ever, when the pipe is being tested or when it is first put in service or else after it has gone through a severe winter and has been subjected to the effects of frost action. It was realized that in the present instance the existing source of supply from the Hudson River could very easily be kept available in reserve for a sufficient period to eliminate such breaks as would be likely to occur in a cast-iron supply conduit and that thereafter such a conduit could reasonably be expected to serve without serious interruption until the construction of the second stage of the project and the duplication of the pipe line.

The year 1928, in which the pipe-line survey was made, was abnormally wet. In connection with the surveys, well-auger borings were made to a depth of 8 ft. or more at frequent intervals, particularly throughout the portion of the pipe-line location extending from the foot of the Helderberg escarpment eastward to Albany, where the surface is generally flat and the material is a fine silt soil which becomes relatively fluid when worked wet.

The bore holes were put down with a hand-auger outfit resembling a posthole auger, and it was found that two men could put down an 8-ft. hole in soil free from stone in from one to two hours. The invert of the pipe, in general, was about 8 ft. below the surface, thus allowing for 4-ft. cover over a pipe 4 ft. in diameter. The surface soil to a depth of about 6 ft. is a fine-textured, brown silt loam. Underneath this is a heavy blue clay, practically impervious and non-water-bearing. The test wells driven during the period of the survey showed that the ground-water table and the surface veneer of silt both follow closely the configuration of the ground surface. In most of the wells the ground-water table on June 27, 1928 was within a foot or two of the surface. As the ground-water level fell, the recession was greatest under the hills.

Following the severe rains which produced the great New England flood of November, 1927, many land slips occurred in this region on steeper slopes. Some of these slips may have been due to a layer of sand above the blue clay, but the test-well borings and subsequent excavation showed that in many instances these land slips occurred on steep slopes where there was no layer of sand, and that in all cases the slippage extended only to comparatively slight depth and generally not even down to the underlying clay. It was found in some instances that these landslides have taken place from time to time in the past, whereas in other instances there had evidently never been any landslide on the slopes affected since the retreat of the glaciers until those which occurred following the flooding rainfall of November, 1927. These circumstances showed the necessity of and pointed the way to the location of the pipe line on hillsides in such a manner as to avoid danger from landslides (1) by avoiding slopes steep enough to be subject to slippage or which had shown evidence of slippage in the past and (2) where it was necessary to cross steep slopes, by running the pipe parallel with the direction of slope and at sufficient depth so that the pipe surface would be below the plane of slippage.

In view of the ground-water conditions observed during the survey, it was expected that unless pipe-line construction was carried out during a very dry year, wet trench conditions would be encountered throughout most of the length of the supply line from the Helderberg escarpment to Albany. This fact had a bearing on the selection of the type of conduit because of the difficulty of making good lead joints in cast-iron pipe under such conditions. It was realized that other types of conduit could be obtained, the jointing of which would not involve this difficulty. Because of the lesser difficulty experienced in making joints on a moist pipe surface, as well as

for economic reasons, it was decided to use Leadite joints. Nature was, however, kindly. The pipe-line construction in the late summer of 1929 and the summer of 1930 was built wholly in dry trench, even in the valley of the Vloman Kill.

As previously mentioned, this pipe line extends for the first section, known as Section 4, a distance of about 44 000 ft. from Alcove Dam to the filter plant through rugged, hilly country where the trench was largely through fissured limestone rock. The second section of the supply conduit, known as Section 5, extends from the filter plant to the Albany city limits at the Normanskill for a distance of 33 000 ft. The largest part of the trench for this section was wholly in earth, the soil consisting generally of a fine silt. The third section of the supply conduit extends from the Normanskill through the city of Albany to Loudonville Reservoir for a distance of about 25 000 ft. Of this portion, known as Section 9, about 50 per cent. was laid through city streets.

In subdividing this pipe line into construction contracts it was realized that the country portion was of a very different nature and would require equipment and methods which would not apply to the construction of the pipe line through the city. It was also realized that Section 4 extended through material considerably more difficult to excavate than Section 5. For these reasons, Sections 4 and 5 were divided into two contracts, known as Contracts 4 and 5, which were advertised for bids at the same time, permitting the contractors to submit a figure for building each section separately or the two sections combined. The low bid on both sections was submitted by Pepper Brothers of Syracuse, N. Y., to whom Contracts 4 and 5 were awarded.

On Section 5 between the filter plant and the Normanskill, the supply conduit passed under three systems of railroad tracks, namely (1) a single-track line of the West Shore Railroad; (2) the Selkirk Yards of the New York Central Railroad consisting of thirteen tracks; (3) a single-track line of the Delaware and Hudson Railroad. It was necessary to adopt special construction where the pipe line passed under these railroad tracks, and it was soon learned that considerable time would be required to reach an agreement with these railroads as to the type of structure to be built, the securing of a right-of-way, and the support of their tracks. In order not to delay the construction of the main pipe line the special construction at the three crossings was included in a separate contract, known as Contract 7.

The third section of the supply conduit through the city of Albany, which was designated Contract 10, was the last portion of the supply conduit to be placed under contract. This was made necessary because of the large amount of work involved in accurately determining the location of underground obstructions which would be encountered in the laying of the line, together with making the necessary studies to determine the points of connection between the supply conduit and the existing distribution system. Contracts 4 and 5 were started in May, 1929, Contract 7 in Oc-

tober, 1929, and Contract 10 in January, 1930. All three of these contracts were completed before December 1, 1930.

It is of interest to compare the average unit prices of the supply conduit applying to those sections located in the country and through the city. The total cost of 77 000 ft. of line from Alcove Dam to the Normanskill was approximately $2 225 000, corresponding to an average unit price of $28.90 per ft. The total cost of the pipe line through the city, which included about 20 100 ft. of 48-in. pipe and 4 300 feet of 42-in. pipe was approximately $877 000, corresponding to an average unit price of $35.90 per ft. This difference in cost is due principally to the slower rate of progress made necessary on construction through the city streets, increased restrictions on backfilling under city streets, and additional cost of repaving.

At all three railroad undercrossings it was necessary to enclose the pipe line in a concrete tunnel structure passing under the tracks. This structure was built of sufficient size to permit the installation and repair of two pipe lines. At the Selkirk Yard, where the structure was 1 100 ft. long, two pipe lines were constructed within the tunnel and cross-connected at each end so that either or both mains could be placed in service. The structure was built to an inside width of 15 ft. and a height of 7 ft. 6 in. These dimensions were sufficient to provide sufficient clearance between the two mains to remove and install pipe sections in case of repair. The structure at the West Shore Railroad was 61 ft. long and at the Delaware and Hudson Railroad 36 ft. long. At the two latter undercrossings only one pipe line was installed as the short length of the structure permitted the line to be easily accessible.

The structure at the Selkirk Yard consisted of a reinforced-concrete floor 24 in. thick, reinforced-concrete sidewalls of gravity type, 5 ft. thick at the bottom and 24 in. thick at the top. The roof of the tunnel consisted of reinforced-concrete slab-units 24 in. thick, 6 ft. 6 in. wide and 18 ft. long. While these slabs were poured in place, they were constructed with joints and lifting hooks so that they would serve as independent units and if necessary could be removed in the future. The total cost of the undercrossing structures exclusive of the cost of pipe was approximately $241 000, corresponding to a total length of 1 197 ft. This cost is equivalent to an average unit price of $201 per ft.

The work of supporting the tracks over the excavation was performed directly by the railroads at the expense of the city. These temporary supports were constructed on timber pile bents. The central span under each track consisted of two 33-in., 202-lb. Bethlehem girders, each 34 ft. long. On each side of the central span were three 12-ft. spans of timber girders consisting of three 8-in. by 16-in. timbers under each rail. The cost of supporting the single track at the Delaware and Hudson Railroad was about $7 100. The total cost of supporting the thirteen tracks at the Selkirk Yard and the single track at the West Shore Railroad was about $60 000, equivalent to $4 300 per track.

The construction of the 48-in. supply conduit through country as performed under Contracts 4 and 5 has proved to be the most interesting feature of the new water supply. The work consisted of laying 15 miles of 48-in. cast-iron pipe through farm lands, across streams, up steep hillsides and through rugged, rocky country. When the work was started in May, 1929, both the engineers and the contractor were greatly concerned as to the possibility of distributing the materials during unfavorable weather conditions and of carrying on continuously the work of excavation and pipe laying during the winter season. The fifteen miles of pipe were laid uninterruptedly during the winter, and this portion of the line was finally completed in the middle of August, 1930. We believe that the contractor's selection of equipment and well planned preparations for distributing the materials were largely responsible for this record of performance.

The pipe for the entire supply conduit extending from Alcove Dam and through the city of Albany to Loudonville Reservoir was furnished under two separate proposals by the United States Pipe and Foundry Company. For the country portion of the line extending for a distance of about 15 miles, this company furnished 35 822 tons of straight cast-iron pipe and 1 053 tons of special castings. For the section of the supply conduit extending through the city of Albany for a distance of about 5 miles this company furnished 11 029 tons of straight cast-iron pipe and 564 tons of special castings.

Delivery of pipe under Contracts 4 and 5 was started about June 1, 1929, and all the materials were received by February, 1930. The right-of-way, provided by the city for the laying of the pipe line, was in general 66 ft. in width. Additional areas were purchased at each road intersection and at intervals along the right-of-way for the storage of pipe and fittings. Although these areas were provided the contractor preferred to haul these materials and distribute them directly along the line of the trench. The pipe trench was located 20 ft. from the right-hand side of the right-of-way (looking toward Albany), leaving a space 46 ft. in width for the distribution of materials. The contractor started hauling and distributing the pipe without making any preparation for temporary roads. Although the season was exceptionally dry, difficulties were encountered during the first week of the work by trucks miring to such an extent that it was decided to build a temporary roadway for the entire length of the line. This temporary roadway was constructed by the use of a road machine shaping gutters on the right or 20-ft. side of the pipe line, the gutters being about 16 ft. apart and the soil being shaped up and graded between them. In especially wet places, brush and saplings resulting from clearing were used as a corduroy mat over which soil was placed to surface the roadway. At these points lateral ditches were dug for drainage. At stream crossings, temporary bridges were constructed of logs and saplings while gates were placed at all fence crossings. Considerable time and money were expended on this preparation, but later results proved it to be justifiable, as the contractor

was able to distribute all his pipe from the time of first arrival to the completion of shipments, directly to the points of use.

The unloading and hauling equipment consisted of a locomotive crane located at the railroad siding and three units of Freeman trucks and trailers. One of these units is shown in Figure 7. Each truck and trailer was mounted on pneumatic-tired wheels, and the unit was equipped with air brakes. All pipe was loaded directly from the cars to the truck and trailer. This was

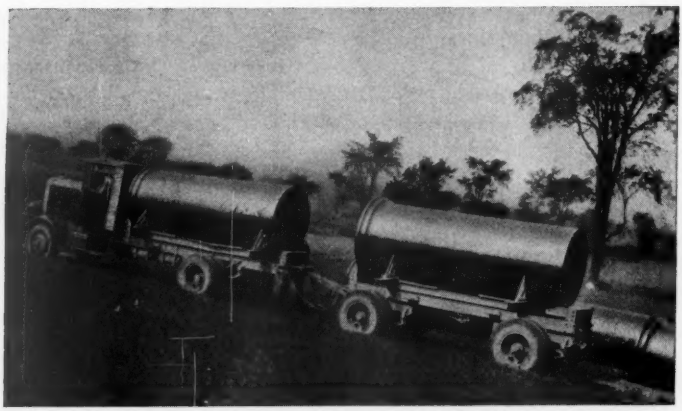

FIG. 7.— FREEMAN TRUCK AND TRAILER HAULING CAST-IRON PIPE.

made possible because the pipe shipments were quite regular in arrival and the cars were usually unloaded on the same day they arrived. During the fall of 1929, with favorable weather conditions and short hauls of two to four miles, as high as thirty trips a day were made with two pipes to each trip, making a total of about 300 tons per day. This performance was, however, the exception, and the average tonnage hauled was approximately 130 tons per day.

Excavation and laying, with few exceptions, were all done by the use of shovels of the hoe type. On the entire contract four shovels were used, as follows: Shovel 1, gasoline engine, 3-cu. yd. capacity, 56 per cent. of the line laid; Shovel 2, Diesel engine, 3-cu. yd. capacity, 21 per cent. of the line laid; Shovel 3, gasoline engine, 1-cu. yd. capacity, 15 per cent. of the line laid; Shovel 4, gasoline engine, 1-cu. yd. capacity, 8 per cent. of the line laid. In general, the excavation and pipe laying were performed by the same machine. The trench was dug by machine for a length of about 12 ft. to within a few inches of grade. Three laborers, stationed in the trench, excavated for block holes and dressed the bottom of the trench to final grade. These laborers also thoroughly cleaned the bell of the pipe already in position. Two blocks were then placed about 6 ft. apart to receive the next pipe section. This pipe, after the spigot end was thoroughly cleaned,

was lifted into the trench with a sling attached to the end of the hoe bucket. Jute was placed around the spigot of the pipe before shoving home, after which the pipe was adjusted for line and grade. It was found that best results for securing proper alignment were obtained by placing the blocking slightly below grade in order that the pipe could be raised to the proper grade, after which thin boards and wedges were placed on top of each block.

Line and grade for the pipe line were furnished by placing offset stakes 25 ft. apart from 8 to 12 ft. off the center line. Levels were taken on the tops of these hubs, and an iron pin about $4\frac{1}{2}$ ft. long was driven opposite each stake. A string was then stretched between these iron pins at an even distance above the top of the pipe. This method served as an automatic check on the levels originally taken and on the computations for setting the string, as a break in grade along the string was easily visible. A spirit level notched out to conform to the outside curvature of the bell was set on top of the bell of the pipe to determine the center line. A rod of proper length and with a plumb bob fastened on one end was used to check the alignment while the pipe was in the sling, and grades were transferred from the string by use of a string level or by the use of a contractor's level.

Where ledge rock was encountered the earth over-burden was first stripped. Drilling was done by using three holes abreast, spaced about $2\frac{1}{2}$ ft. on centers in both directions, and all holes were drilled to a depth of 2 ft. below the bottom of the pipe, which resulted in the removal of rock to a depth of 1 ft. below the bottom of the pipe. After the rock was broken up, it was levelled off by the shovel operating on timber mats and was then removed in the same manner as earth. The earth and rock were separated on opposite sides of the trench in order that the earth could be used for backfilling around the pipe. Where this earth was not sufficient in amount for backfill, additional material was borrowed from areas designated along the right-of-way.

Each pipe-laying organization consisted of one foreman, three laborers on top of the ditch or trench, three laborers in the trench and three to five laborers placing and tamping the backfill around the sides and for a depth of 1 ft. on top of the pipe. In addition, one man was provided to handle Leadite, together with one operator and one oiler on each shovel. The maximum length of line laid in one day by one machine was performed by Shovel 1. This equipment laid 36 lengths on two occasions. Table 3 shows the average performance of each shovel on pipe laying, based on actual days worked.

Shovel 1 operated under the most ideal conditions since its work was largely on Section 5, where 85 per cent. of the excavation was in earth. Shovel 4 had similar operating conditions, 90 per cent. of the excavation being in earth. This machine, however, was of smaller capacity than Shovel 1. Shovel 2 operated under the most severe conditions where 60 per cent. of the trench was in rock. Shovel 4 was operated at scattered locations, being used principally on making excavation at all the creek crossings.

TABLE 3.— SHOVEL PERFORMANCE.

Shovel No.	Actual Days Worked.	Total Length Laid, Ft.	Average Laid per Day, Ft.
1	301.5	42 510	141.0
2	181	15 925	88.0
3	169	11 067	65.5
4	58	6 329	109.1
All Shovels	709.5	75 831	106.9

Data have previously been given as to the average length of main laid by each piece of equipment. Records were kept of the total time devoted by each machine to the contract work, and this time was classified as to days devoted to pipe laying, days devoted to other work, days lost for repairs and days lost on account of bad weather. Table 4 gives a summary of this information.

TABLE 4.— SHOVEL PERFORMANCE.

Shovel No.	Total Number of Days Devoted to Contract.	Percentage of Time on Laying Pipe.	Percentage of Time on Other Work.	Percentage of Time on Repairs.	Percentage of Time Lost on Account of Bad Weather.
1	382½	76	6	7.2	10.8
2	225	80	9	2.0	9.0
3	274½	62	28	2.5	7.5
4	68½	85	6	0.7	8.3
All Shovels	950½	75	12	4.2	8.8

Leadite was used as a joint material for the entire length of the 48-in. supply conduit. Special care was used in cleaning the bells and spigots of the joints before placing the pipe and seeing that the jute was properly rammed home. Shortly after the work started, it was found advisable to backfill the pipe immediately to a depth of 1 to 2 ft. over the top of the pipe in order to protect the line from extreme temperature changes. These precautions are in our opinion largely responsible for the excellent results secured on final leakage tests and also account for the small number of defective joints that were developed. For the entire fifteen miles only two joints blew out completely and a third joint blew out for a length of about 18 in. The two joints to blow out completely were the result of testing caps or plugs moving due to insufficient bracing. The third joint which blew out was poured in a very wet section in winter, and it was more than likely that the joint was not properly cleaned. On making joints in the pipe line, one joint was always left unpoured in the trench in order to avoid possible damage due to the jarring of the line when the next pipe was placed.

On Contracts 4 and 5, where the entire line was outside the city, the pipe was tested in sections ranging from one-third of a mile to one mile in length, and the pressure applied corresponded to a level 50 ft. above the spillway of Alcove Dam for Section 4 and 50 ft. above the elevation of the filter-plant clear-well for Section 5. These pressures averaged from 100 to 150 lb. per sq. in. The specifications required for preliminary tests a leakage not to exceed 300 g.p.d. per in. diameter per mile of pipe. Table 5 gives the results obtained on preliminary and final tests on Sections 4 and 5.

TABLE 5.— RESULTS OF PIPE TESTS, SECTIONS 4 AND 5.

Section Number.	Length, Miles.	Allowable Leakage, g.p.m.	Actual Leakage.		Date of Test.
			g.p.m.	Gal. per Inch-Mile per Day.	
Preliminary Tests, Section 4.					
1	.860	8.60	4.49	157	Jul. 23, 1930
2	.972	9.72	7.06	218	Apr. 14, 1930
3	.309	3.09	3.20	310	May 1, 1930
4	.301	3.01	2.64	263	Jun. 2, 1930
5	.962	9.62	7.34	228	Aug. 30, 1930
6	.680	6.80	11.74	518	Aug. 30, 1930
7	1.256	12.56	4.46	107	Aug. 30, 1930
8	1.111	11.11	4.37	118	Aug. 30, 1930
9	.777	7.77	7.48	289	Aug. 30, 1930
10	.899	8.99	6.95	232	Jul. 31, 1930
Total 1–10	8.127	81.27	59.73	220	
Final Tests, Section 4.					
1–9	7.220	72.20	19.788	82	Oct. 26, 1930
10	.983	9.83	4.68	143	Nov. 8, 1930
1–10	8.203	82.03	24.468	89	
Preliminary Tests, Section 5.					
1	0.562	5.62	4.49	239	Sept. 14, 1929
2	1.115	11.15	9.55	257	Nov. 1, 1929
3	0.705	7.05	8.16	347	Nov. 1, 1929
4	0.929	9.29	12.18	393	Dec. 23, 1930
5 and 6	1.876	18.76	11.60	185	Apr. 30, 1930
7 and 8	1.065	10.65	6.93	195	May 12, 1930
Total 1–8	6.252	62.52	52.91	254	
Final Test, Section 5.					
Total 1–8	....	67.905	3.74	16.5	Oct. 8, 1930

From Table 5 it will be noted that the total of preliminary tests on Section 4 resulted in an average leakage of 220 g.p.d. per inch-mile while the final test showed an average leakage of 89 g.p.d. per inch-mile; likewise, the total of the preliminary tests on Section 5 showed an average leakage of 254 g.p.d. per inch-mile while the final test showed an average leakage

of 16.5 g.p.d. per inch-mile. In comparing these results it is interesting to note the elapsed time between the making of preliminary and final tests. On Section 5, the time of making the final test averaged 8 months after making the preliminary tests while on Section 4, the time of making the final test averaged 2.9 months after the preliminary test. These results indicate the strong probability of considerable reduction in leakage in Section 4 after this portion of the line is subjected to pressure for an additional period of five or six months.

It is interesting to note that in about 15 miles of 48-in. cast-iron pipe laid under Contracts 4 and 5 not a single length developed defects or breakage after installation in the line. One fitting was found damaged due to the operations of another contractor. On Contract 10 where 5 miles of 42-in. and 48-in. pipe were laid through the city, one length of pipe developed a defect under test while another length developed a break under test, which apparently was the result of damage by the contractor in hauling.

The principal structures along the 15 miles of the 48-in. supply conduit outside the city of Albany included 43 entrance manholes, 28 blow-offs and 27 air valves. The entrance manholes consisted of a flanged casting with bolted cover, enclosed in a brick manhole. The blow-offs were provided with a cast-iron tee to which was bolted a flanged gate valve. These blow-offs were constructed at low points in the line to provide quick drainage for the emptying of the line. Each blow-off was installed in a brick manhole, and the necessary vitrified or cast-iron drain was laid to a nearby stream. Air valves were installed at principal high points along the line, each air valve being housed in a brick manhole.

With approximately one hundred structures of these various types scattered across country through a distance of 15 miles it was realized that considerable difficulty might be experienced particularly in winter in locating and identifying the brick manholes at a time when the ground would be covered with snow. For this reason reinforced concrete posts 8 in. square and 8 ft. long were provided which were planted 4 ft. in the ground alongside of each structure. On each post were cast symbols, such as BO12, indicating the location of Blow-off 12. During the present winter it has been necessary to inspect and operate these structures, and these concrete post markers have made it possible to locate each manhole without delay and without the necessity of referring to records or references.

Clearing and Grubbing the Reservoirs. The clearing and grubbing of Alcove and Basic Creek reservoirs was combined under one contract. As previously mentioned, the reservoir on Hannacrois Creek formed by Alcove Dam will constitute the main impounding reservoir for the city. This reservoir, with flow line at el. 618, will have an area of water surface at this elevation amounting to 1 440 acres. At Alcove Reservoir, all wooded growth was cleared and grubbed for a vertical distance of 7 ft. above the flow line or to el. 625 and for a vertical distance of 18 ft. below the flow line or to el. 600. The area within this margin amounted to 600 acres. Below

el. 600, the wooded growth was cleared but not grubbed, and this area of clearing amounted to 984 acres.

In the operation of the new system, Basic Creek Reservoir will not be used for full-time storage as an impounding reservoir but will be used only for diversion purposes and for storing the excess flood flow of Basic Creek until such time as the stream subsides and this water can pass through the tunnel into Alcove Reservoir. With this method of operation, Basic Creek Reservoir will be filled only a small percentage of the time, and the area behind the dam will be exposed the greater part of each year. For this reason it was deemed unnecessary to grub any of this area so that this contract included only the clearing of this reservoir up to an elevation 5 ft. above the flow line, which included an area of 322 acres.

In presenting information to the contractor as to the character of the work to be performed the character of the growth was classified under two main headings, namely: (1) wooded area, (2) non-timbered area. The non-timbered areas were those devoted largely to cultivations where only scattered trees existed. The wooded areas included those portions of the land covered by heavy brush or timber.

On the contract drawings, the various wooded areas were given an identification number, and a table was prepared showing the density and character of the wooded growth. Each area was given a density index representing the average number of trees in an area 20 ft. wide by 100 ft. long. This density index was applied to the wooded growth consisting of trees over 8 in. in size. This character of growth was classified as Type 1. Wooded growth was further classified as Types 2 and 3. Type 2 represented trees between 2 in. and 8 in. in size while Type 3 represented all growth under 2 in. in size, consisting principally of brush and saplings. Density of vegetation as classified under Types 2 or 3 was described as light, medium or heavy.

With this information shown on the contract drawings, together with locations of all houses and buildings to be removed, this contract was advertised in April, 1929 and was awarded to Winston & Company of Kingston, N. Y., on June 6, 1929. The specified date of completion was March 1, 1930 in order to coördinate this work with the filling of Alcove Reservoir which was scheduled for January 1, 1930. The contractor actually completed this contract on December 31, 1929.

The following are the principal unit prices which applied to this contract:

	Cost per acre, Alcove Reservoir.	Basic Creek Reservoir.
Clearing and grubbing all wooded areas..	$403.00	....
Clearing all wooded areas.....	95.00	$100.00
Clearing and grubbing non-timbered areas	50.00	....
Clearing all non-timbered areas.....	20.00	50.00
Final clearing.....	15.00	15.00

While the above costs are given on a unit-price basis, the bids were not received in this manner. The contractor was furnished with approximate areas applying to each item together with the location of these areas as shown on the contract drawings, and he was required to submit a lump-sum bid for the total area applying to each item. It is believed that the classification of the various types of work together with the location of these areas on the map was of considerable help to the contractors in preparing their bids with the result that the city secured the benefit of a very favorable low bid. The method of payment also worked out to advantage since there were no disputes during the life of the contract as to the character or quantity of work for which the contractor was to receive payment.

Highway Reconstruction. Basic Creek Reservoir inundates only one town road which is of minor importance with respect to traffic. This road is used principally by the scattered farm residents on the west side of the reservoir. Before Basic Creek Dam was constructed the flat area above the dam was subject to flooding during high flow of Basic Creek, and this road was normally covered with water one or more times each year, usually in the spring.

The engineers for the Board of Water Supply proposed to the town-road authorities that this road be raised 4 ft. throughout its length of about 2 300 ft. which would place the road surface 4 ft. below the crest of Basic Creek Dam. Stream-flow records applicable to Basic Creek for the past fifteen years indicated that at this higher elevation the road would have been flooded only three or four times during this period of fifteen years, with the result that under the new conditions interruption to traffic would be considerably less than formerly. The town-road authorities recognizing the light traffic use and present poor condition of this road, entered into an agreement with the Board of Water Supply to permit them to raise the grade of the road and to construct a new water-bound Macadam road 16 ft. in width with 5-ft. shoulders on each side, which was a material improvement over the existing highway. Existing parallel roads were available both on the north and south sides of the reservoir.

It was at first thought that the creation of Alcove Reservoir would present considerable difficulties in finding road facilities to replace the existing roads. The principal problem existed in the high promontory of land situated between the two projecting arms of the reservoir on the south side. Within this area were approximately twenty farms totaling about 2 000 acres. These farms were of low value consisting principally of wood lots worth from $20 to $25 per acre. Allowing these farms to remain, the occupants would have been completely shut off from the state road on the north side of the reservoir, and the city would have been obligated to construct an expensive bridge across Alcove Reservoir at a cost of from $100 000 to $150 000. It was quickly seen that the purchase of these farms was highly desirable both from the viewpoint of economy and for the purpose

of securing additional sanitary protection for the reservoir. This policy resulted in the purchase of approximately 4 050 acres of land for Alcove Reservoir, the water surface of which covers an area of only 1 440 acres.

With the acquisition of this land it was possible to close about 8 miles of town roads which were used purely for local needs. Fortunately, no large centers of population exist near Alcove Reservoir. The principal traffic is in east and west directions along the state road extending from Ravena on the east to Dormansville and Westerlo on the west. That portion of the state road flooded by the new reservoir was relocated and rebuilt around the northern side. This work included the construction of about 3 miles of bituminous-Macadam state highway of a 16-ft. width at a total cost of approximately $186 000. A large portion of this cost was attributable to an embankment about 25 ft. high and about 1 200 ft. long across the north arm of the reservoir and containing about 60 000 cu. yd. of fill.

The principal local traffic which makes use of the town roads in this vicinity travels between the state road in the vicinity of Alcove in a southwesterly direction to the town of Greenville. A portion of this road was inundated by the south arm of Alcove Reservoir, and for this reason the Board of Water Supply agreed with the town-road authorities to improve, widen and relocate about $3\frac{1}{2}$ miles of highway at a cost of approximately $88 000.

Considering the acquisition of approximately 4 500 acres of land for the two reservoirs and the closing of more than 8 miles of road, it is believed that the expense of road relocation amounting to approximately $288 000 is unusually small.

Cemeteries. The sanitary regulations covering the operation of the new water supply prohibit the burial of a human body within 300 ft. of a reservoir or within 100 ft. of a water course. This resulted in the necessity of the removal of existing graves from those cemeteries within the reservoir area before the flooding of these reservoirs and the removal of other graves from cemeteries located along water courses before the supply was placed in use. On the Albany project, there existed one large public cemetery, known as Indian Fields Cemetery, containing about 200 graves, and eleven small private cemeteries located on individual farms. Negotiations were conducted with the Trustees of Indian Fields Cemetery and with the owners of private cemeteries resulting in the creation of a new cemetery located downstream from Alcove Dam. This new cemetery, containing about $6\frac{1}{3}$ acres was graded, fenced and provided with roads and was so laid out as to permit the transfer of all bodies from all cemeteries to this single location. This work was started the latter part of October, 1929, and substantially completed by February, 1930, although several additional months were required to complete the erection of headstones, the building of roads and the seeding and fertilizing of the ground.

This work was performed on a unit-price basis under the following items:

	Original Quantity.	Final Quantity.	Unit Price.
1. Opening and refilling graves	469	721	$7.59
2. Furnishing boxes and placing remains in boxes	469	692	12.24
3. Transporting and reinterring remains	469	687	7.16
4. Removing, transporting and resetting small stones	676	628	0.98
5. Removing, transporting and resetting large stones and monuments	47	37	30.00
6. Furnishing, marking and setting stone markers	46	294	9.00
7. Concrete foundations for stones — cu. ft.	2 500	1 091	0.48
8. Building brick or stone vaults	20	6	50.00

It will be noted that there was a considerable increase in the amount of work as shown by the final quantities compared with the original estimate. It was originally planned to remove bodies from twelve cemeteries. After the work progressed, bodies were removed from three additional cemeteries which were located on land acquired by the Board of Water Supply but which did not require removal under the sanitary regulations. These three cemeteries, however, were located along roads which were subsequently closed to traffic, making it advisable to remove these bodies to a new location.

Many of these cemeteries were extremely old, and some of the private cemeteries had been abandoned for generations. There were many graves without markers or means of identification. Under these circumstances it was extremely difficult to locate and to predict the existence of the graves in advance of actual construction. The specifications provided for exploratory excavation to determine the existence of graves and this method, together with the prodding of the ground with one-inch iron bars, made it possible to discover a large number of graves of which there was no evidence on the surface of the ground.

Before work was started on removal of bodies from an existing cemetery, all known graves were accurately located by measurement and were identified by number. A plot was reserved in the new cemetery of approximately the same size, and in practically all cases the arrangement of the graves at the new cemetery was the same as in the existing cemeteries. The contractor simultaneously excavated new graves along with the opening of existing ones. Wooden boxes properly tagged with identification numbers were placed alongside of each grave, and when the skeleton was uncovered an inspector was notified before the remains were disturbed. In practically all cases the remains consisted only of a skeleton, and frequently only a few bones were found in each grave. Each box was prepared with a bed of excelsior and the skeleton was removed by starting at the feet and by placing each bone in the box in its proper order. It was originally planned

to use boxes 30 in. wide by 7 ft. long by 18 in. high. It was soon found, however, that most of the remains were so disintegrated that smaller boxes could be used with the result that a majority were furnished 18 in. wide by 3 ft. long by 12 in. high.

The requirements of the specifications with respect to reburial were rigidly enforced. These requirements necessitated the reburial on the same

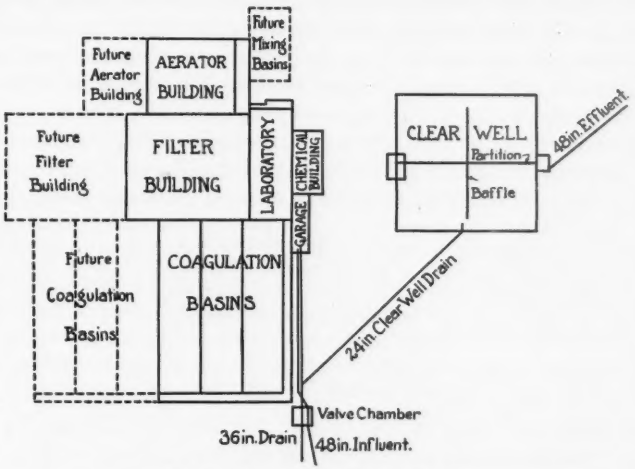

FIG. 8.— LAYOUT OF FILTER PLANT.

day as the opening of the graves in order that there might be no possibility of improper identification. The headstones for each grave were tagged and marked with an identification number and these were later erected at the new cemetery. A separate form was used for each grave giving a complete history, including date and time of opening grave, name of workman and inspector, time of reburial, identification and inscription on stones, names of relatives present, location of old grave, location of new grave.

After this work was completed plats were prepared showing the location of all graves in existing cemeteries including identification number and plats of all graves in the new cemetery with the corresponding number. These plats together with the individual records for each grave were turned over to the Board of Water Supply in duplicate so that they could be placed on file with the proper public authorities.

Filter Plant. The construction of the new filter plant was carried out in the usual manner by preparing a general contract covering the building of main structures, including basins, filters, foundations, buildings and pipe work and by preparing additional contracts for other items of work which could be carried out satisfactorily under a separate contract and which otherwise would be performed as a subcontract. An outline layout of the various units of the plant is shown in Figure 8. It is impracticable to describe

in detail this portion of the new water supply since it is deserving of discussion in a separate paper. The general plan, however, shows that the raw water enters the plant from the south and passes through the chemical building to the aërator. After aëration the water passes through the mixing basins located in the basement of the washwater building where it is treated with chemicals before entering the coagulation basins. On leaving the coagulation basins the water is filtered, after which it is chlorinated on passing through the chemical building to the clear well. Provision is made for lime treatment before entering the clear well, which is designed as a sedimentation basin. The 48-in. supply conduit extends from the outlet of the clear well to the city of Albany. The general plan indicates a scheme for future extensions of all units providing for an ultimate average capacity of 64 m.g.d. as compared with a present constructed capacity of 32 m.g.d. The following is a brief summary of the basic elements of design of this plant.

Normal capacity — m.g.d. (Present construction)	32
Overload capacity — m.g.d. (Present construction)	40
Normal capacity — m.g.d. (Future development)	64
Overload capacity — m.g.d. (Future development)	80
Mixing-basin detention period — min.	20
Length of travel in mixing basin — ft.	1 200
Velocity of flow in mixing basin — ft. per sec.	0.9
Loss of head in mixing basin — ft.	.3
Coagulation-basin detention period — hr.	2.5
Coagulation-basin velocity — ft. per min.	1.32
Filter-unit capacity — m.g.d.	.4
Number of units	8
Size of each unit — ft.	46.5 x 30.5
Thickness of sand — in.	30
Thickness of gravel — in.	18
Normal rate of wash — in. per min.	24
Washwater storage — gal.	252 000
Elevation washwater tanks above sand — ft.	26.5
Clear-well capacity — m.g.	.2

As previously noted and as shown in Figure 1, the filter plant is located along the 48-in. supply conduit at a point where the ground elevation is close to hydraulic grade and about midway between the source of supply and the city of Albany. With this arrangement, all the flow through the plant is by gravity. This location of the plant is rather unusual in that it is situated on a plateau along the edge of the Helderberg escarpment where practically all the excavation is in rock. This rock is a limestone, seamy in character, with beds from 2 to 4 ft. in thickness and with a dip of about three degrees. The city of Albany was fortunate in securing extremely low unit prices for this rock excavation. The contractor bid $1.40 per cu. yd. for general excavation work and $4.00 per cu. yd. for trench excavation work, which in some instances was as deep as 26 ft.

On the general excavation work, drill holes were made 6 to 8 ft. deep

and 4 ft. on centers, and these were loaded to about one-half depth with 40-per cent. dynamite. After the rock was loosened, it was removed by shovels and transported to storage piles for later use in embankments or disposed of in a spoil pile located in a wooded area behind the filter plant. This method of blasting proved too violent in removing rock to vertical faces but was satisfactorily controlled with respect to depth. The dip of the rock planes made it impossible to excavate to a uniform level so that irregularities had to be refilled with foundation concrete.

On trench excavation, the contractor used more careful methods with very satisfactory results. The main drain, which is about one mile in length and which was provided to convey washwater to a nearby stream, was located adjacent and parallel to the 48-in. supply conduit previously laid by another contractor. Practically all this drain trench was in rock, being 23 ft. away from the 48-in. pipe line for a majority of the distance while at a location near the filter plant it was only 13 ft. away. For this trench, three lines of holes were drilled 3 to 4 ft. deep and about 30 in. on centers both ways, and these holes were loaded lightly with dynamite. The charges were fired by delayed shots, the center holes being fired first, then the holes on the far side of the supply-conduit trench and lastly the holes next to the supply-conduit trench. When the supply conduit was laid, an earth backfill was provided both under and over the pipe. Throughout this mile of drain-trench excavation, there was only one point of damage to the 48-in. supply conduit. This occurred where a concrete anchor was poured around a one-sixteenth bend of the 48-in. pipe. These results demonstrated that the method of blasting the drain trench together with the cushioning effect of earth backfill were sufficient to prevent damage to the 48-in. cast-iron main.

Loudonville Distribution Reservoir. The new distributing reservoir which is now nearing completion is situated at the northern limits of the city of Albany at an elevation sufficient to provide adequate pressures for the high-service district, being about 50 ft. higher than the present Prospect Reservoir. By referring to Figure 1, it can be seen that the supply conduit enters the city from the south, completely crosses the city approximately through its center and delivers water into the new reservoir at the northerly limits.

There is an important advantage in having, as in this case, the supply come in from one side of the city, with the equalizing reservoir located on the opposite side. Water will be taken directly from the conduit to supply the residential and higher-level districts, and water will also be taken through pressure-reducing valves to supply the downtown business and congested-value districts, where the present maximum pressure of 100 lb. per sq. in. will be maintained. The filter plant will operate at a uniform rate throughout the 24 hours. During the day, the excess of consumption over average daily use will be supplied from the distributing reservoir, while during the night the excess of supply over demand will refill the reservoir, which will thus act simply as a balancing reservoir. With an excessive draft such as

might result from an unusual conflagration and during the peakload period of each day, water will flow into the principal feeders of the distribution grid from both directions. The velocity in the main supply conduit under these conditions will be less than it would be with the same total supply coming from one direction only. Since the friction loss varies as the square of the velocity, the result of feeding from two directions will be to cut down the fluctuations in pressure throughout the gridiron of the system to a fraction of what they would be with a feed from one direction only. Put in another way, a supply from two directions is equivalent in some cases to doubling the capacity of the principal supply mains in the distribution system.

Figure 9 shows that the Loudonville Reservoir is divided into two basins by a center dike, Basin A having a capacity of 75 m.g. and Basin B having a capacity of 25 m.g. Since this reservoir is of the balancing type a single 48-in. pipe line is brought into a gatehouse located in the center dike from which the water is discharged into either or both basins through channels leading out from the gatehouse. By means of a check-valve arrangement of the pipe within the gatehouse, the outlet from each basin is taken from the far end through 36-in. pipes, thus providing circulation during intervals of inflow and outflow. These 36-in. pipes, through valve connections in the gatehouse, are connected to a 24-in. drain some 3 600 ft. long which discharges into an existing 7-ft. city storm-water drain. This drain can be used for overflow purposes or for the emptying of either basin.

The site of the reservoir consists of a natural elongated kettlehole at the highest elevation within the city limits. This depression, apparently of glacial origin, consists of a series of three connected hollows, which are about 65 ft. deep. The underlying soil is a sandy loam with a small percentage of gravel. The two depressions which are being developed at present and which are designated as Basins A and B will hold 100 m.g. Immediately adjoining Basin B to the north is a third depression designated as Basin C which was purchased by the city of Albany and which can be developed to provide an additional storage of 100 m.g.

The topographical features of this site are of considerable advantage in reducing the amount of excavation as compared with that normally required to construct a reservoir of this size. The character of the soil required the placing of an impervious lining. In order to construct this lining it was only necessary to excavate sufficiently to uncover material suitable for the support of the lining and to secure sufficient additional material to construct the embankments at the center and ends of the depressions. The excavation, which varied from 1 to 3 ft. in depth, was so graded as to provide level areas for the bottoms of the basins and uniform planes for the side slopes. The total excavation required for embankments and including the stripped material which could not be used for this purpose, amounted to 118 000 cu. yd.

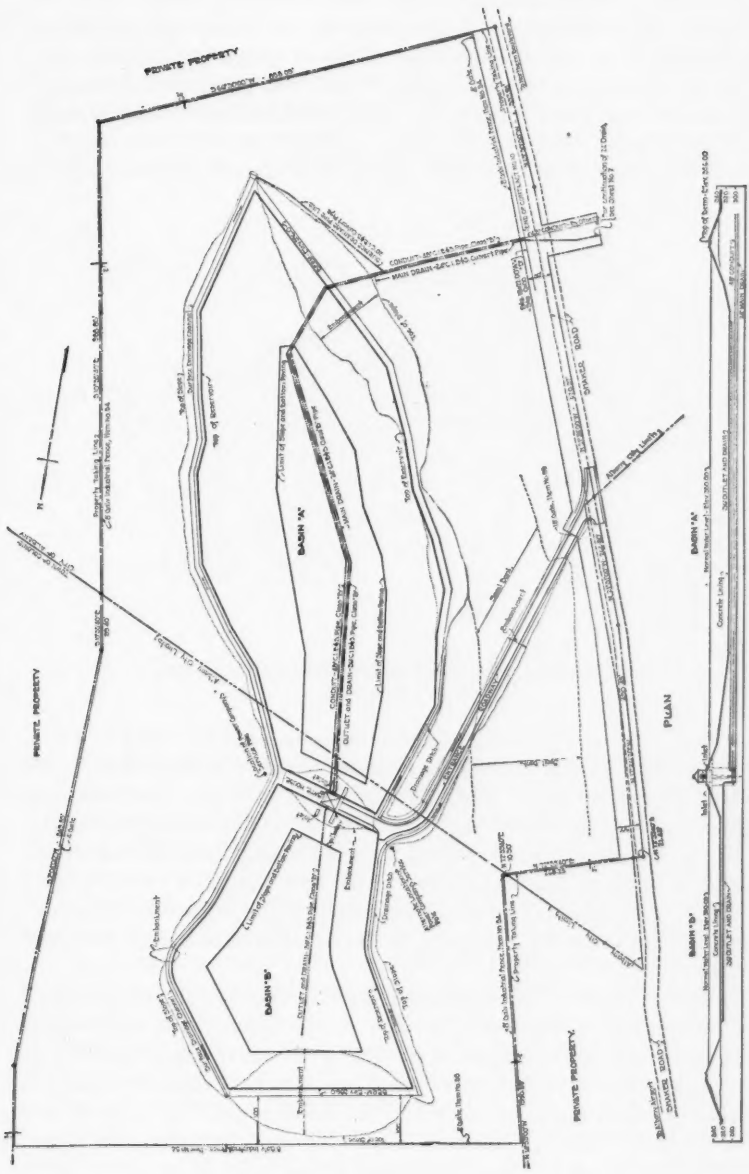

FIG. 9.— LOUDONVILLE DISTRIBUTING RESERVOIR.

The bottoms of the depressions were so permeable that water never stood in them to an appreciable extent, even after heavy rains.

Under these circumstances, the lining of the basins was one of the most important as well as most costly items of design and construction. After a careful study of various methods of reservoir lining heretofore used, it was decided that for the reservoir bottom a two-layer type of construction would prove most effective. As shown in Figure 10, each 4-in. layer in Basin B was composed of units 20 ft. square. The joints between each 20-ft.

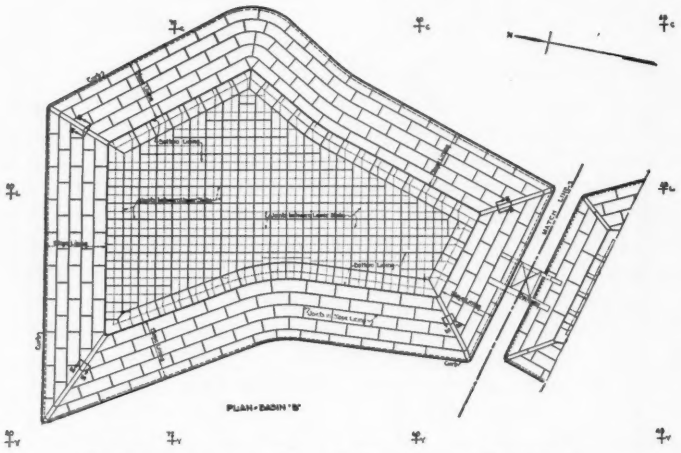

FIG. 10.— LINING OF BASIN B, LOUDONVILLE RESERVOIR.

slab were composed of premoulded bituminous material $\frac{3}{8}$ in. thick and 4 in. deep, and the joints of the top layer were staggered with respect to the joints of the bottom layer. The reinforcement in each 4-in. layer was composed of a welded steel fabric with mesh 6 in. by 6 in. and weighing 0.85 lb. per sq. ft. Before placing the second layer, a continuous membrane waterproofing of two-ply construction was placed over the top of the first layer and in addition, under all joints between slabs of the top layer of concrete, an extra ply of fabric 3 ft. wide and an extra mop coat of asphalt were also applied.

The side slopes of these basins had an average inclination of 1 on 3 while the maximum slope was 1 on $2\frac{1}{2}$. It was realized that considerable difficulties would be experienced in placing two layers of concrete each 4 in. thick, and a single-layer type of construction of greater thickness and with carefully designed joints was adopted for the side slopes. The type of construction used is shown in Figure 11. The lining on the slopes, which was placed to a thickness of 8 in. was arranged in slab units 16 ft. wide by 40 ft. long. Each unit was reinforced with a steel fabric weighing 1.70 lb. per sq. ft. and with mesh 6 in. by 6 in. These slab units were constructed suc-

cessively in rows starting at the bottom and in each row alternate slabs were first placed, followed by the placing of intermediate ones. The joints, as designed, provided for a concrete shoulder which would underlap the lower edge of the adjacent slab. A crimped copper strip 10 in. wide and weighing 24 oz. per sq. ft. was placed continuously along all joints, and this strip was filled with a premoulded bituminous joint filler $\frac{1}{2}$ in. thick and 8 in. deep. Along all joints $\frac{3}{4}$-in. dowels, 18 in. long, were placed 18 in. on centers, one end of each dowel being provided with a sleeve to permit free movement of

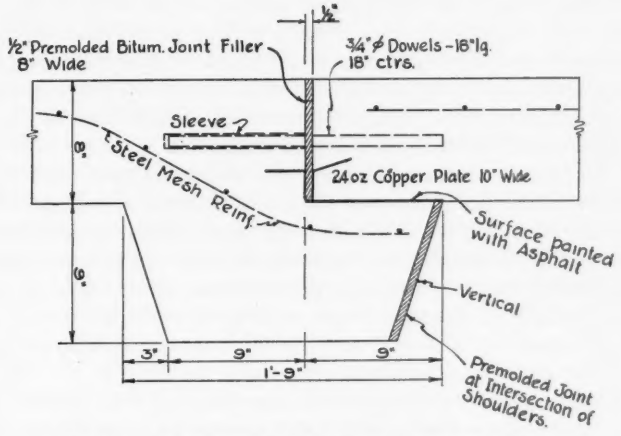

FIG. 11.— CONSTRUCTION DETAILS OF CONCRETE LINING, LOUDONVILLE RESERVOIR.

each unit for contraction or temperature changes. The use of these dowels is similar to the method employed for joints on concrete-road construction, and they were placed for the purpose of preventing unequal vertical movements of adjacent slabs. As shown in the details of Figure 11, the top surface of the concrete shoulder was painted with asphalt to facilitate free horizontal movement of the adjacent slabs. Construction of Basin A was similar to that of Basin B.

The contractor experienced no difficulty in the construction of these joints. When the first slab was poured the copper strip was placed on a wooden form of the same size and in the same position as the $\frac{1}{2}$-in. premoulded joint filler. The $\frac{3}{4}$-in. dowels were placed in this form with the sleeve on the inside of the form. Before the adjoining slab was placed, the dowels were withdrawn from the sleeve and the form removed. The joint filler was then placed inside the copper strip, and holes were punched in the joint filler to match the location of the sleeves. The $\frac{3}{4}$-in. dowels were then replaced in the sleeves, the top of the shoulders painted with asphalt and the adjacent slab poured.

The specifications for the lining required that the concrete should develop a strength in 28 days ranging from 2 500 to 2 700 lb. per sq. in. and that the mix should be not leaner than a proportion of 1:2:3½. The contractor decided to use concrete from a central mixing-plant. The aggregates proposed were analyzed as to grading and percentage of voids, and strength tests were made for varying proportions, quantity of water and degree of slump. The coarse aggregates used consisted of a crushed gravel varying from ¼ in. to ¾ in. in size for the 4-in. slabs and varying from ¼ in. to 1 in. in size for the 8-in. slabs. In order to place the concrete satisfactorily on the slope, it was decided that the slump should not exceed 4 in. A trial mix in the proportion of 1:2:3½ demonstrated that these proportions resulted in a large excess of cement-sand mortar above the voids of the coarse aggregates, and for a given slump this excess of mortar required a larger quantity of water per bag of cement. With this proportion, sufficient water to provide the desired slump resulted in deficient strength test.

An analysis of the voids of the fine and coarse aggregates together with the results of experiments, showed that the mix could be properly changed to the proportion of 1:1.8:3.75. This proportion furnished a sufficient excess of cement-sand mortar above the amount of voids in the coarse aggregates and at the same time required for a given slump a smaller amount of water per bag of cement. This proportion, as adopted, made it possible to use 7 gal. of water per bag of cement and to develop a strength of approximately 2 700 lb. per sq. in. in a 28-day test.

All the concrete for this reservoir was furnished by a central mixing-plant located about one mile distant and was hauled to the job in trucks of the end-dump type. The contractor considered various methods of handling and placing the concrete on the slopes, including the use of crane, tower and chutes, and conveyor belts. For the placing of the bottom it was possible to deliver the concrete in trucks directly to each slab panel. For the slopes, the concrete was delivered around the top rim of each basin. Wooden troughs about 2 ft. square were placed on the ground leading down the slopes. The concrete was discharged into these troughs, and laborers with shovels and hoes pushed the concrete along the trough into the slab panel. Because of the uniform grading of the aggregates and the slump adopted, the contractor experienced no difficulty in placing and holding the concrete to the desired slope. It was shoveled in position and struck off with a screed, operated on top of the side forms. After screeding, the surface was then finished by hand with wooden floats and steel trowels.

In preparing the contract drawings, considerable thought was given to the design and method of construction of this lining, after which the arrangement of the slab units was adopted as indicated in Figure 10. It is interesting to note that over 90 per cent. of this lining has been placed, and the location and arrangement of the slabs as constructed is in exact conformity with the layout as shown on the contract drawings.

Distribution System Improvements. The present water supply of Albany

is delivered entirely by pumping from the Hudson River into two distributing reservoirs. Bleecker Reservoir with flow line at el. 242 serves the low-service district while Prospect Reservoir with flow line at el. 297.5 serves the high-service district. The controlling elevation for the new supply will be fixed by Loudonville Reservoir with flow line at el. 350 and minimum draft line at el. 330. This additional elevation over Prospect Reservoir is much needed to provide satisfactory pressures in the high-service district. The scheme of operating the new supply provides for the use of Loudonville Reservoir for high-service distribution, the use of Prospect Reservoir for low-service distribution, and the use of Bleecker Reservoir as a reserve in case of fire.

The rearrangement of these reservoirs together with the abandonment of the pumping system and the use of the new 48-in. supply main, which completely crosses the city, required considerable alteration to the existing distribution system. This work was taken care of under Contract 20.

The 48-in. supply conduit was connected to all the main feeder lines of the high-service district at each point of intersection. Additional feeder mains were also constructed. Connections were also made from the 48-in. conduit through pressure regulators to supply mains connected to the Bleecker and Prospect reservoirs. In addition to this construction, nine pressure regulators were installed on high-service mains at the boundary between the high-service and the low-service districts. These regulators were uniformly distributed throughout the city, so that this method of supply should tend to equalize pressures throughout the low-service and to maintain a more uniform pressure, since each valve can be set to provide for the local demand in that portion of the low-service district which it serves.

All the connections made under this contract were so designed that the pumping system can continue to function until the new filter plant and reservoir are ready for operation. When this stage is reached various sections of the city can be divorced successively from the pumping system and supplied by the new gravity system.

The introduction of a new source of water supply, the water having different chemical characteristics from the existing supply, with important changes in pressure and with reversals in the direction of flow of water in many of the pipes in the gridiron system, is a matter that requires careful consideration. In order that there shall be the minimum annoyance to the users of water while the change is taking place, a detailed study has been made of the entire distribution system so that the water can be introduced gradually, permitting the blowing-out of mains to clean out sediment and establish the new flow conditions with the least possible interruption of service. The plan, as a result of this study, will be used as a basis of introduction of the new supply to the city of Albany in the spring of 1931.

RECENT DEVELOPMENTS IN CHLORINATION.

BY FRANCIS D. WEST.*

[*Read September 23, 1930.*]

One interested in sanitation may read today many articles on the subject of pre-chlorination. The author thinks he may claim, without fear of contradiction, to have been one of the first to experiment with chlorine for this purpose, though, strange as it may seem, his results were contrary to those obtained by the proper application and regulation of that chemical.

In the early part of 1909,[1] at the Torresdale Filter Plant, Philadelphia, Pa., Joseph Siddons, as plant superintendent, and the author, as chemist, endeavored to sterilize raw Delaware River water as applied to one of 120 preliminary or roughing filters. For this purpose, an ordinary (and very inefficient) laundry cell was used to generate sodium hypochlorite, and apply it to the influent to the filter. The bacterial efficiency thus obtained was low, and the turbidity in the effluent appeared to be higher than that of the other 119 filters. After experimenting for about three months, the plan was abandoned because the tools were too crude.

As stated in 1915 in a paper[2] before the Convention of the American Water Works Association, recourse was then had to chloride of lime and later, to liquid chlorine. In this paper, "Disinfecting 200 000 000 Gallons of Water Daily," the author endeavored to show, by a careful analytical study of some three years' work, that liquid chlorine as a disinfectant was far superior to chloride of lime. The ratio was given as 6:1 whereas, theoretically, it should be about 3:1.

It is the author's conviction that the disinfecting action of chlorine is the same, pound for pound, whether applied as a gas, as a solution in water, or as a compound with calcium, or sodium, in the form of hypochlorites. Everything depends on the thoroughness with which the chlorine is mixed with the water or sewage to be treated. This statement should be qualified by saying that alkalinity seems to have some influence, alkaline solutions being apparently more efficient than acid ones.

There appear to be three theories of the action of chlorine on bacteria. Leal, testifying in the Boonton, N. J., case in 1911, according to Race,[3] advanced the nascent-oxygen theory; bacteria being destroyed by oxidation. For a long time this theory was generally accepted, although Phelps, testifying at the same trial, suggested that hypochlorites were directly toxic to microörganisms. He gave no proof, but this second theory seems to have since been accepted by most investigators. Braidech,[4] at Cleveland, and several others have proved conclusively that the nascent-

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oxygen theory can only account for a part of the reduction in biochemical oxygen-demand.

In his discussion of Mohlman's[5] paper, Enslow states: "It is evident from the Chicago studies that the reduction in oxygen demand cannot be attributed to the oxidative action of the chlorine. Two parts of chlorine produce but one part of oxygen, whereas the average reduction in oxygen demand at Chicago indicates that one chlorine atom would have to furnish two oxygen atoms, if the oxidation theory could be applied. In other words, the oxygen-demand reduction was four times that theoretically accounted for by the nascent-oxygen hypothesis. In all probability the efficacy of chlorination in reducing oxygen demand can best be explained by the known reactions of chlorine on nitrogenous (amido) compounds to produce chloro-substitution products — apparently useless as bacterial foods and therefore less putrescible." Enslow also mentions the fact that some of these amido compounds are antiseptics.

As to the third theory, Bunau-Varilla,[6] when speaking of the action of sodium hypochlorite on bacteria, says:* "The molecule of hypochlorite of sodium in attacking organic matter must emit rays, the action of which is analogous to ultra-violet rays on microscopic life, creating around the scene of chemical action an extended zone of destruction of this microscopic life." Bunau-Varilla gives the results of several experiments with quartz tubes to prove his theory, and shows further, from his experience, that the disinfecting action of chlorine is greater in the dark.

One thing seems evident, if chlorine can be prevented to any extent from combining chemically with matter other than bacteria, its efficiency as a germicide will be enhanced. Proper use of ammonia seems to be a step in this direction.

In his analysis of the results reported by the author on experiments with liquid chlorine *versus* chloride of lime, Race,† in discussing the 1:6 ratio, as well as the 1:8 ratio of Kienle[7] or the 1:6 ratio of Catlett, states: "West used hypochlorite solutions abnormally strong, 3.6 to 10.4 per cent. available chlorine, which would increase the difficulty of extracting all soluble hypochlorite, and, further, at best only 87 per cent. of the available chlorine was extracted." Hale,[8] who at first obtained results similar to the author's, later reported he got, under favorable conditions, a ratio nearer the theoretical of about 1:3. The increased efficiency of liquid chlorine in the author's work was due to the ability to diffuse the chlorine through the water being treated in a more thorough manner than was possible with the crude method of handling chloride of lime.

Chloride of lime has fallen into disuse largely due to the difficulties of handling, storing and maintaining the full strength of the chemical. Sodium hypochlorite finds some use today especially in small water supplies and swimming pools. One of the chief objections to its use is that at

*Pp. 73 and 82.
†Page 99.

best one can only concentrate it into a solution with about 10 to 12 per cent. available chlorine; this solution, if not properly made, breaks down rapidly and is expensive to transport. The French appear to be using it more than we do. Bunau-Varilla gives an account of the use on a large scale of this chemical for the city of Rheims. The claim is made that doses of 0.1 p.p.m. are employed in place of the 1 to 4 p.p.m. of chlorine previously applied. Bunau-Varilla states, in part: "Of these infinitesimal doses, nothing portrays to the taste the presence of chlorine, and the antisepsis of the water delivered for human consumption is absolute." He also makes another interesting observation: "As a result of our experiments, we state that the amount of chlorine taken up by a water containing a definite amount of organic matter varies with the amount of chlorine added."

The author is fortunate in being permitted to present (in advance of publication by the author, Harry Johnson, Chemist in Charge of the Sewage Disposal Plant at Flint, Mich.) some studies on "Super-chlorination."

Studies have shown that the amount of residual chlorine carried in a treated water rather than the total amount of chlorine applied is the controlling factor in measuring the efficiency of sterilization. If the distribution of chlorine in a body being treated is inadequate, over-chlorination of some portions of the body results. The excess chlorine reacts chemically with the humus matter, forming permanent chlorine or "separable chlorine compounds," which have little disinfectant action. Therefore, it is an economical practice to adjust the distribution of chlorine for maximum efficiency.

Johnson made a careful study of the subject of super-chlorination using for his work 0.2- and 0.3-per cent. aqueous solutions and dry gas with a sewage that had a chlorine demand of 9 p.p.m. He plotted his results and shows that there is very little difference between 0.2- and 0.3-per cent. solutions, but that sewage will take up more chlorine as a dry gas. He noted that no two sewages gave the same super-reaction and that greater efficiency with chlorine could be obtained with some sewages than with others. Johnson emphasizes the importance of a thorough distribution of the chlorine in the body being treated. This is a point that appears to a large degree to have been overlooked.

Bunau-Varilla* shows that, by agitation, the efficiency of disinfection is markedly increased. He says, in part: "Agitation that seems at first to be of secondary importance, seems to be of utmost value." E. Techoueyres[9] states: "Bactericidal action is found to be better if the water is agitated during and for some time after the addition of chlorine." It is only logical to assume that to bring about a chemical reaction with a gas such as chlorine and a liquid such as water, or sewage, that the more they are agitated, the more intimate the mix and the more efficient the chemical action. Race† likewise calls attention to this fact. There should be no question of the advisability of a thorough mixing.

*Page 47.
†Page 43.

Mention has been made of the possibility of delayed or deferred action to prolong the antiseptic action of chlorine and prevent it from using up its bactericidal powers by acting as nascent oxygen and oxidizing organic material other than microörganisms.

Ammonia has come to the fore recently, though it is hard to understand why we waited ten years before adopting the ideas presented by Race in his experiments on chloramines at Ottawa. Perkins[10] states that for ten years he has been using ammonia with chlorine to maintain a chlorine residual in an open-air pool exposed to sunlight at a temperature of 80° F. The formation of chloramines in sewage by the action of chlorine on ammoniacal compounds may account for increased and prolonged bactericidal action of chlorine in sewage disinfection.

Berliner[11] makes some interesting suggestions about the use of ammonia on sewage. He criticizes the old method of Wanklyn — free and albuminoid ammonia determination — which dates back to 1885, stating that in 45 years we have not changed nor improved it. Berliner says: "There are a number of basic materials in sewage that yield to direct and steam distillation and which would, under ordinary analysis, be reported as free ammonia,— indoles, skatole, peptone, decomposition products, carbamates, volatile amines, hydrazines, etc." For those interested, a study of the use of ammonia with chlorine for sewage treatment may prove of value in attempts at increased efficiency.

From what the author can gather from a study of the available literature, chlorine as a disinfectant appears to be more efficient in alkaline solution. There is one process that makes use of a tower containing limestone through which the aqueous solution of chlorine is passed, the purpose being the neutralization of the hydrochloric acid formed. It might prove advisable in certain cases to add to the water sodium hydroxide or carbonate. That this hydrochloric acid may be considerable is shown by Ellms[12] who observed a reduction of 2 p.p.m. of alkalinity for each part of chlorine.

If ammonia is used and chloramines are to be formed, the water must be alkaline. Ammonia should be added first and thoroughly mixed with the water before the chlorine is added.

A further study of the use of chlorine as hypochlorite of sodium should be made. Such a study should include the possible use of chloramines. Berliner has informed the author that he has made 0.1-per cent. solutions that are stable at normal temperatures in the sunlight.

At Cleveland, when treating filtered water with ammonia and chlorine, the ammonia is added but 40 seconds before the chlorine. It is thoroughly mixed with the water, however, as the water falls down a cascade before the chlorine is applied. Several reports are to the effect that ammonia-chlorine treatment is efficient in reducing algæ.

As to the relative amounts of chlorine and ammonia, it may be stated, that in dealing with clear or filtered waters, to maintain a chlorine residual

of 0.1 p.p.m. with chlorine alone, there should be introduced 0.3 to 0.7 p.p.m. chlorine. In the ammonia-chlorine process, for a like residual, 0.1 to 0.2 p.p.m. of ammonia with 0.15 to 0.25 p.p.m. chlorine is sufficient. Records of plants using the combined ammonia-chlorine treatment show a ratio of about 3 parts of chlorine to 2 parts of ammonia. In almost every instance there is a reduction in the amount of chlorine required.

In connection with the use of ammonia with chlorine, a word of caution is advisable. Ammonia has been used to eliminate phenol taste, tastes due to over-chlorination and also those due to algæ and other vegetable growths. In using ammonia, operators have found it possible to reduce the amount of chlorine. An excess of ammonia can do no harm, however, so there is some tendency to use crude, home-made devices for controlling the amount of ammonia. These devices are wasteful of ammonia, especially in small plants. While the small plant operates 24 hours per day, there is no supervision at night and with a makeshift apparatus it is quite conceivable that no ammonia would be applied at times when supervision is not available. If the amount of chlorine is cut down, it means that during these periods there will be insufficient chlorine used to properly sterilize the water. This may result in an epidemic. It is absolutely imperative, if ammonia is going to be used with chlorine, that the amounts of ammonia be controlled just as carefully as we now control chlorine and that properly designed machines be used for this purpose.

Ornstein,[13] in Germany, has published some interesting results with chlorine-copper for algæ. He passes a portion of the discharge from his chlorinator tower (using the absorption-tower method) through a container holding copper chips. The chlorine solution dissolves a certain amount of copper as chloride. Ornstein reports the following: "The effectiveness of chlorine as an algicide can be greatly increased by traces of copper. Killing of algæ was accomplished with 2.5 p.p.m. Cl and 0.18 p.p.m. Cu where doses of 20 p.p.m. Cl or doses of 100 p.p.m. $CuSO_4$ failed."

Chlorine is said to assist coagulation. Thus, in their Patent No. 1 733 070 on the use of chlorine in paper manufacture, Pattillo and McMahan state:

> In white water which we have coagulated, we have found a pH value of 5.5 to 5.0 most effective for coagulation and that the coagulation is more effective in the presence of a small amount of residual chlorine which appears to serve as a catalyst in promoting the coagulation.

A careful study shows that chlorine, when used ahead of the hot-process lime-soda systems, prevents the formation of silicate scale. This is true also to a lesser degree when chlorine is introduced into the make-up water for a boiler ahead of such compounds as sodium aluminate.

Among the many uses for chlorine is that of sterilizing new mains. For this purpose a portable chlorinator should be used. Portable chlorinators are of value as emergency machines or as spare units. They are

useful also for experimental work with the many possible applications such as pre-chlorination and in a chloro-boat for algæ control.

Further studies should be made not of how much chlorine should be used but how little can be used to get the desired results, namely, a safe water and minimum operating costs. In this work, proper and thorough mixing are of great importance.

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Discussion.

Linn H. Enslow.* There is not much that one can add to this excellent and condensed contribution. The author has covered the ground very well, and the points which he has brought out are open to little discussion. I might say one or two things which occur to me.

The Bunau-Varilla theory of killing bacteria by rays emitted by hypochlorite (somewhat analogous to ultra-violet rays) does not appear to have been confirmed by recent investigators. As to just how or why an apparently effective bacterial destruction can be secured by the addition of such small dosages of sodium hypochlorite has not been revealed. Likely enough, the ammonia content of the water being treated may prove to be an important factor and efficient distribution of the very dilute hypochlorite solution another.

Now I should like to say something in the nature of a warning in connection with the use of the ammonia-chlorine process which has gone forward so rapidly. One popular argument for its adoption is the reduction in chlorine consumption when preammoniation is practiced. What I have to say might be expected as a consequence of my position with the Chlorine Institute, but I should be compelled to say as much even though I was representing the manufacturers of ammonia. As a matter of fact, the hydrogen gas that was formerly wasted in the manufacture of chlorine is now converted to ammonia and sold by chlorine producers at a profit.

I am leading up to the warning that it may not be safe practice to curtail the chlorine dosage applied as at first may seem feasible. There is already abundant evidence to indicate that a higher concentration of residual chlorine should be carried in waters treated with ammonia than has been found necessary prior to the introduction of this new treatment. The residual chlorine present in ammoniated water is there in the form of compounds having a milder action than chlorine itself, or other chlorine compounds of higher oxidation potential — stronger germicidal power in short periods of contact.

It is assumed that chloramines are produced in the ammonia-treated water. An upward change in pH value reduces the effectiveness of chloramines. So does a drop in temperature. With a normal concentration of residual chlorine, a materially longer contact period is required at pH values above 7 and at temperatures below 20° C. to produce efficient sterilization. To offset these conditions it may be necessary to increase the residual chlorine being maintained in the water fifteen minutes after treatment to a point in excess of quantities found adequate when chlorination alone is practiced. Fortunately, this can be done without production of chlorine taste and odor in the water.

Until one has adequately proved that sterilization is satisfactory in the

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period of contact available between the point of chlorination and the delivery of the water to consumers, one should proceed cautiously in the matter of maintaining the same residual chlorine values as formerly applied. Further, the chlorine residual necessary with ammonia treatment should be checked up during decreasing temperature of the water. That residual should be maintained which is found necessary at 0° C. for the available period of contact. The same warning applies where waters having pH values in excess of 7 are to be dealt with. Cutting the cloth too fine when it comes to saving chlorine may prove a very false, if not dangerous, economy.

I jotted down another note of interest in connection with chlorine as an algicide. Ornstein has developed a process in which a portion of the total chlorine applied is converted into copper chloride by passing it through a vessel filled with scrap copper. The mixture of copper chloride and chlorine is then applied to the water. The increased effectiveness of the chlorine brought about by the presence of minute quantities of copper seems phenomenal. The combination seems to be much more effective than either chlorine or copper salts applied singly in relatively large quantities. These observations made in Germany have been confirmed in America. The copper-chlorine process for algæ control and for eliminating biological "slimes" from streams, cooling towers, paper mills and swimming pools would seem to have a future, particularly in situations where the chlorine demand of the water is extreme.

Reference is made by the author to possible advantages from adding ammonia to sewage for the purpose of producing chloramines, as suggested in a paper by Berliner. Rideal about 1906 called attention to the fact that chloramines were produced in sewages as the result of chlorination and also pointed out the advantages of chloramines in procuring a persistent germicidal effect. Rideal used bleaching powder as the chlorinating agent, but no ammonia was involved other than that naturally present in the sewage.

In recent experiments conducted at the Baltimore, Md., sewage-treatment plant it has been shown that an important factor in producing chloramines in sewage is the alkalinity — or more properly, the pH value — of the sewage *at the point of chlorine application*. Bleaching powder solutions are quite alkaline, and to secure a similar condition with chlorine it has been found practical and economical to merge together in a pipe line the discharge from a chlorinator and an emulsion of lime from a dry-feed machine. The mixture flows continuously to the sewage. The lime used is sufficient to convert all the chlorine to hypochlorite and to leave an excess of 10 or 20 per cent. to produce a temporarily increased pH zone in the sewage at the point of application of the alkaline mixture.

It has been shown that in the Baltimore sewage, chloramines are readily produced by this process from the ammonia naturally present in the sewage. The reduction in the amount of chlorine necessary to maintain residual chlorine more than pays for the lime used. Incidentally, the residual chlorine secured persists for a much longer period than when chlorine alone

is used. At Baltimore, this condition is particularly desirable for the purpose of carrying residual chlorine into the tidal estuary receiving the plant effluent.

The reaction between the chlorine solution and lime emulsion is completed practically instantaneously; therefore no period of retention for the reaction need be considered. The ratio of lime to chlorine is 1.12 to 1.25 lb. of hydrated lime to each pound of chlorine used. The mixture should produce a temporary pink "flash" when phenolphthalein indicator is used. As the sewage to be treated in some instances may already be quite alkaline, the use of the lime-chlorine process may not prove so advantageous. In instances where the sewage is abnormally low in alkalinity (low pH value), however, the proportion of lime to chlorine may even be profitably increased above the 1.25:1 ratio.

THE 1929–1930 DROUGHT IN NEW ENGLAND.

BY H. B. KINNISON.*

[*Read December 10, 1930.*]

Since the beginning of life upon this planet, water has been the most necessary and valuable of all minerals. It is essential not only for the support of all living things, but also for nearly all of man's present activity. Water was undoubtedly first used by man for domestic purposes only, but he has since found it a necessity for navigation, water power, sanitation, irrigation and industry. The first use man made of water is still the most important. The general health and prosperity of any community requires that there be a dependable supply of pure water available at all times. Should a community's water supply fail or prove inadequate, that community at once becomes an undesirable place in which to live. In general, other conditions being equal, the community with the most dependable supply of water of desirable quality, will be the first to develop. One factor that will limit the extent of the future growth of our centers of population will be the quantity of dependable water that can be made available.

The truth of these statements can be seen from the fact that our early cities and centers of industry have developed where the supply of water was abundant and unquestionable.

One of the most serious considerations confronting the engineer in the development of a water supply for a community is the dependability or adequacy of the supply during periods of drought. In the study of this phase of the question, it is highly desirable that there be available for the stream to be used for a water supply a long period of discharge data. These data should be of sufficient length to cover at least one, and preferably several periods of extreme low flow, such as occurs during periods of drought. If such records of stream flow are not available the next best step is to consult data of similar nature on other nearby streams, preferably of similar character. In doing this, considerable care and judgment should be exercised since the yield of drainage areas may vary to a much greater extent than a superficial examination might indicate. The nature and depth of the surface soil and the imperviousness of the subsoil should be carefully considered as well as the ruggedness of the drainage area, the slope of the stream and the nature and extent of the vegetation on the drainage area.

Due to the wide range in percentages of precipitation collected by the streams, precipitation data over the drainage area being studied should be used with great caution. Such studies, however, will usually indicate the relative yield of the stream for various periods under consideration.

An opportunity to determine the adequacy of our water supplies was

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recently given us by the occurrence in southern New England of one of the most severe droughts in history. Although rainfall for the month of October and November, 1930, in Massachusetts was up to normal or above, it is not certain that the drought has yet been permanently broken.

Beginning with the month of June, 1929, rainfall in southern New England became deficient, and this deficiency steadily increased, with but few exceptions, until by the first of July the rainfall during the year ended had been but about 70 per cent. of the normal. By the end of December,

TABLE 1.— RAINFALL FOR PERIODS DURING SEASONS OF DROUGHT.

Location.	Year Ending.	Rainfall in Inches.	Per Cent. of Average.
Average for Massachusetts.	May 31, 1930	32.87	74
Lowell, Mass.	Jun. 30, 1930	26.62	60
	May 31, 1915	27.22	61
	Sep. 30, 1883	28.12	63
	Jun. 30, 1911	28.29	64
	May 31, 1836	24.90	56
	Dec. 31, 1846	28.03	63
Wachusett Watershed.	Jun. 30, 1930	29.48	65
Smallest in 29 years.	May 31, 1925	35.51	79
	Ave., 33 yrs.	45.15	100
Sudbury Watershed.	Jun. 30, 1930	30.78	69
	May 31, 1925	37.19	84
	Dec. 31, 1909	37.91	85
	Dec. 31, 1910	35.90	81
	Dec. 31, 1911	35.46	80
	Ave., 55 yrs.	44.48	100
Concord, N. H.	Jun. 30, 1930	25.79	67
	Dec. 30, 1894	27.64	71
	Dec. 30, 1899	28.57	74
	Dec. 30, 1908	26.25	68
	Dec. 30, 1913	29.04	75
	Dec. 30, 1924	28.65	74
	Ave., 72 yrs.	38.80	100
Burlington, Vt.	Jun. 30, 1930	32.73	101
	Dec. 31, 1854	25.45	78
	Dec. 31, 1873	25.92	80
	Dec. 31, 1879	24.27	75
	Dec. 31, 1880	25.26	78
	Dec. 31, 1882	25.64	79
	Dec. 31, 1894	22.96	71
	Dec. 31, 1908	23.49	72
	Dec. 31, 1913	25.75	79
	Dec. 31, 1914	22.62	70
	Dec. 31, 1915	25.68	79
	Ave., 94 yrs.	32.55	100
St. Johnsbury, Vt.	Jun. 30, 1930	30.84	88
	Dec. 31, 1894	27.15	77
	Dec. 31, 1904	29.67	85
	Dec. 31, 1909	29.78	85
	Dec. 31, 1921	27.32	78
	Ave., 35 yrs.	34.86	100

TABLE 1.—(*Continued*).

Location.	Year Ending.	Rainfall in Inches.	Per Cent. of Average.
Boston, Mass.	Jun. 30, 1930	29.89	66
Cambridge.	Dec. 31, 1762	24.47	54
	Dec. 31, 1822	27.20	60
	Dec. 31, 1828	32.41	71
	Dec. 31, 1837	33.52	74
	Dec. 31, 1846	29.95	66
	Dec. 31, 1887	33.75	74
	Dec. 31, 1899	34.69	76
	Dec. 31, 1902	33.93	75
	Dec. 31, 1905	32.08	71
	Dec. 31, 1908	30.07	66
	Dec. 31, 1910	28.33	62
	Dec. 31, 1912	34.56	76
	Dec. 31, 1914	34.31	73
	Dec. 31, 1918	34.40	76
	Dec. 31, 1923	34.91	77
	Dec. 31, 1924	34.21	75
	Ave., 96 yrs.	45.46	100
Springfield, Mass.	Apr. 30, 1930	27.35	63
	Dec. 31, 1864	30.90	71
	Dec. 31, 1905	31.02	71
	Dec. 31, 1912	31.17	71
	Dec. 31, 1924	31.12	71
	Ave., 81 yrs.	43.76	100
Hartford, Conn.	Sep. 30, 1930	36.65	85
	Dec. 31, 1914	34.30	79
	Dec. 31, 1921	34.50	80
	Dec. 31, 1924	33.54	77
	Ave., 50 yrs.	43.30	100

1929, this deficiency amounted to 6.50 in. for the state of Massachusetts — and by the end of September, 1930, the deficiency had increased to 16.65 in.

The Water Resources Branch of the U. S. Geological Survey studies the fluctuations of the rivers at about 120 points in New England where daily or continuous stages of the rivers are obtained at regular stream-flow measuring stations. These stations are well distributed over New England, being located at advantageous points on all the main rivers and many of their principal tributaries. From these records the mean daily flows, the average monthly flows, the average yearly flows, as well as the minimum and maximum for each month of the year and for the year as a whole, are obtained for each station.

On examination of the records it was found that the river stages remained at or above normal during the spring of 1929 due to a period of ample precipitation immediately preceding. During June, 1929, the stages dropped at a faster rate than usual and by early summer were at unseasonably low stage. An exceedingly dry summer and fall caused the river stages to drop even more, so that before the usual fall increase in river flow occurred, the streams had reached extremely low stages. The fall rains failed to reach their normal amount and the snowfall in southern New England

was extremely light. As a result, we experienced no high spring freshets in 1930, only medium stages being recorded in our rivers at the time when our heavy spring run-off is expected. In northern New England average snowfall was experienced, and although the spring freshets were not high because of weather conditions, the flow in the rivers during the spring and early summer was well sustained, and the summer flows were about normal.

Rainfall. Table 1, in which is given the amount of yearly rainfall during periods of low precipitation at various stations where long-time records have been obtained, shows that the precipitation at Burlington, Vt., was somewhat greater than normal for the year ending June 30, 1930. The Merrimac and Connecticut rivers which rise in this northern section have not shown a tendency to drop below the average low-water flow.

In Lowell, Mass., however, the rainfall for the year ending June 30, 1930 amounted to but 26.62 in., which is but 60 per cent. of the normal rainfall. This is the lowest rainfall experienced since 1836 when a yearly total of 24.9 in., or 56 per cent. of normal, was recorded, which was the minimum since 1826, when the record was started. These precipitation data indicate that the third driest year was for the year ending May 31, 1915 when the precipitation was 61 per cent. of normal. For the year ending September 30, 1883, one of the most severe droughts in history, the precipitation was 63 per cent. of normal as it was also during the year ending December 31, 1846.

Rainfall data on the Sudbury and Wachusett watersheds may be obtained by averaging the precipitation at four stations on each watershed. The average precipitation over 33 years on the Wachusett Watershed is 45.15 in., but for the year ending June 30, 1930 only 29.48 in. had been received, or only 65 per cent. of the normal. These figures are nearly duplicated for the Sudbury Watershed, the percentage of normal rainfall received for the year ending June 30, 1930 being 69.

Considering the state of Massachusetts as a whole, the precipitation for the year ending May 31, 1930 indicated the lowest yearly rainfall of 32.87 in. and was 74 per cent. of the average. The oldest precipitation records obtainable in Massachusetts are for Cambridge, Mass., and date back to 1749. During this entire period the lowest yearly precipitation was during the year 1762 when the total amounted to but 24.47 in. or 54 per cent. of normal. The precipitation for the year ending June 30, 1930 was 29.89 in. or 66 per cent. of normal. In looking through this record, which is practically continuous since the middle of the 18th century, there appear to have been 17 separate years when the precipitation amounted to less than 75 per cent. of normal, which indicates that we can expect a drought in which the yearly precipitation amounts to but 75 per cent. of normal, once in about ten years. The drought recorded for 1930, however, is of considerably less frequency, since the precipitation was but 66 per cent. of normal, and probably occurs more nearly once in twenty-five or thirty years.

Stream Flow. We are far less fortunate in the matter of stream-flow

data than in precipitation records. The science of stream gaging, however, appears to have originated and passed through its early development stages in New England so that we have available in New England some of the longest records of discharge in the United States. Table 2 compares yearly discharge of various streams for periods of low flow.

At only three points, all located in eastern Massachusetts, are records of discharge available for the droughts of 1879–1883, 1908–1912, and 1929–

TABLE 2.— RIVER DISCHARGE FOR YEARLY PERIODS DURING SEASONS OF LOW FLOW.

Station.	Year Ending.	Yield in sec.-ft. per sq. Mile.	Per Cent. of Average.
Nashua River, at Clinton, Mass.	Jun. 30, 1930	.786	46
	Jun. 30, 1911	.79	47
	Dec. 31, 1911	1.055	63
	Nov. 30, 1917	1.24	73
	Mean, 33 yrs.	1.689	100
Cochituate River, near Cochituate, Mass.	Jun. 30, 1930	.65	45
	Oct. 30, 1871	1.03	72
	Jan. 31, 1881	.71	49
	Nov. 30, 1883	.72	50
	Jan. 31, 1893	.93	65
	Feb. 28, 1895	.91	63
	Mar. 31, 1911	.64	44
	Jul. 31, 1911	.59	41
	Dec. 31, 1880	.727	50
	Dec. 31, 1883	.719	50
	Dec. 31, 1894	.951	66
	Dec. 31, 1908	.964	67
	Dec. 31, 1911	.871	60
	Mean, 67 yrs.	1.44	100
Sudbury Watershed, Mass.	Jun. 30, 1930	.66	44
	Dec. 31, 1880	.895	60
	Dec. 31, 1883	.824	55
	Mean, 56 yrs.	1.498	100
Merrimack River, at Lawrence, Mass.	Jun. 30, 1930	1.00	68
	Sep. 30, 1883	.942	64
	Sep. 30, 1895	1.00	68
	Sep. 30, 1911	.786	53
	Mean, 51 yrs.	1.48	100
Housatonic River, near Great Barrington, Mass.	Jun. 30, 1930	1.20	63
	Sep. 30, 1915	1.36	71
	Mean, 17 yrs.	1.91	100
Connecticut River, at Orford, N. H.	Jun. 30, 1930	1.42	80
	Sep. 30, 1911	1.29	73
	Sep. 30, 1915	1.34	76
	Mean, 29 yrs.	1.77	100
Souhegan River, at Merrimack, N. H.	May 31, 1930	.877	53
	Sep. 30, 1911	.839	51
	Mean, 21 yrs.	1.64	100
Ware River, at Gibbs Crossing, Mass.	Jun. 30, 1930	.748	48
	Sep. 30, 1925	1.02	65
	Mean, 18 yrs.	1.56	100

TABLE 3.— COMPARISON OF DRY PERIODS BASED ON AVERAGE DISCHARGE.

Consecutive M'ths.	Cochituate. Sec.-ft. per sq. Mile.			Sudbury. Sec.-ft. per sq. Mile.			Merrimac. Sec.-ft. per sq. Mile.			Nashua. Sec.-ft. per sq. Mile.	
	1879–1883.	1908–1912.	1929–1930.	1879–1883.	1908–1912.	1929–1930.	1879–1883.	1908–1912.	1929–1930.	1908–1912.	1929–1930.
1	0.213	0.155	0.121	0.133	0.031	0.006	0.266	0.266	0.311	0.087	0.109
2	.321	.202	.181	.212	.068	.014	.344	.297	.344	.165	.170
3	.337	.219	.212	.237	.106	.059	.374	.343	.391	.192	.194
4	.332	.252	.229	.246	.118	.080	.437	.371	.418	.178	.238
5	.339	.276	.269	.259	.136	.159	.453	.389	.432	.252	.303
6	.391	.305	.288	.261	.154	.227	.491	.391	.457	.311	.360
7	.406	.317	.320	.282	.213	.215	.567	.424	.515	.403	.439
8	.437	.328	.356	.310	.287	.292	.588	.431	.569	.468	.509
9	.470	.454	.488	.388	.378	.460	.618	.503	.614	.589	.602
12	.779	.756	.757	.827	.612	.732	.921	.783	.915	.792	.786
15	.800	.819	.658	.864	.605	.642	.858	.712	.814	.678	.723
18	.787	.660	.610	.795	.626	.573	.825	.763	.770	.750	.706

NOTE: For Cochituate and Sudbury basins discharge corrected to yield from land area only.

1930, which are the three most severe droughts on record. These points are located on the Cochituate River at Cochituate, Mass., Sudbury River at Framingham, Mass., and the Merrimac River at Lawrence, Mass. It has been possible to make a comparison of the yield of these drainage areas for the period of these three droughts. The results of this comparison are shown in Table 3 and in Figs. 1–3. In order that the discharge from the Sudbury and Cochituate drainage areas could be compared directly, it was found desirable to correct the yield from the total drainage area to yield from land surface only. This was accomplished by determining the yield from the water surface only, the remainder of the yield coming from the land surface. The yield from the water surface was taken as the difference between the monthly rainfall and the corresponding evaporation for that month as determined by FitzGerald.* This table indicates that on the Sudbury and Cochituate watersheds the drought following 1929 supplied less water per sq. mile than either of the other two for various consecutive months studied. On the Merrimac River, however, the 1929–1930 drought is found in the middle position throughout the period stated. In general, it can be deduced, therefore, that the drought following 1929 was considerably more severe than the drought following 1879, and was about equal in severity to the drought following 1908.

The yield on the Wachusett Watershed uncorrected for evaporation from water surface (see Fig. 4) for the year ending June 30, 1930 was .74 sec.-ft. per sq. mile, or but 44 per cent. of normal, as compared to a rainfall which was 65 per cent. of normal. The previous lowest year in the 33 years of record was for the year ending June 30, 1911 and was .79 sec.-ft. per sq. mile, or 46 per cent. of normal.

*"Rainfall, Flow of Streams, and Storage" by Desmond FitzGerald. Transactions Am. Soc. Civil Engineers, Sept., 1892.

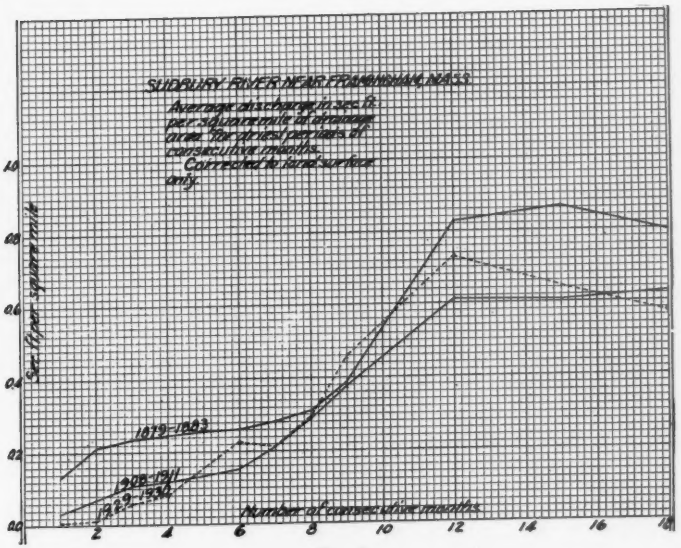
SUDBURY RIVER NEAR FRAMINGHAM, MASS.
Average discharge in sec. ft. per square mile of drainage area for driest periods of consecutive months.
Corrected to natural surface only.
Run-off in sec. ft. per sq. mile
1.0
0.8
0.6
0.4
0.2
0.0
1879-1883
1908-1911
1929-1930
Number of consecutive months
2
4
6
8
10
12
14
16
18

FIG. 1.

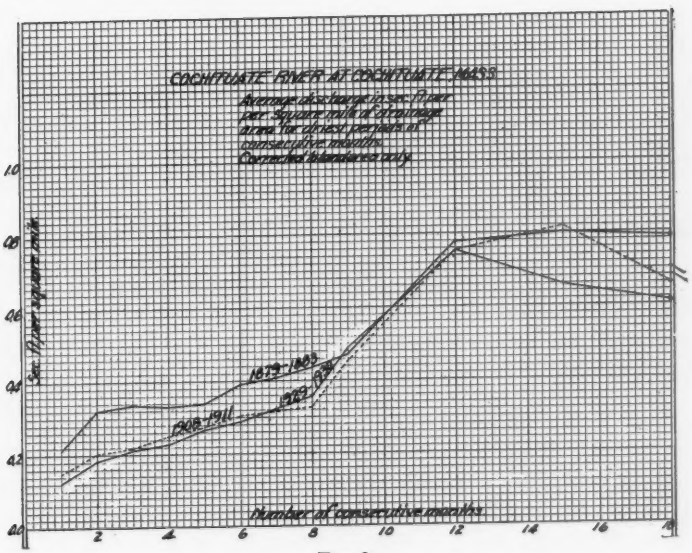
COCHITUATE RIVER AT COCHITUATE, MASS.
Average discharge in sec. ft. per square mile of drainage area for driest periods of consecutive months.
Corrected for natural flow only.
Run-off in sec. ft. per sq. mile
1.0
0.8
0.6
0.4
0.2
0.0
1879-1883
1908-1911
1929-1930
Number of consecutive months
2
4
6
8
10
12
14
16
18

FIG. 2.

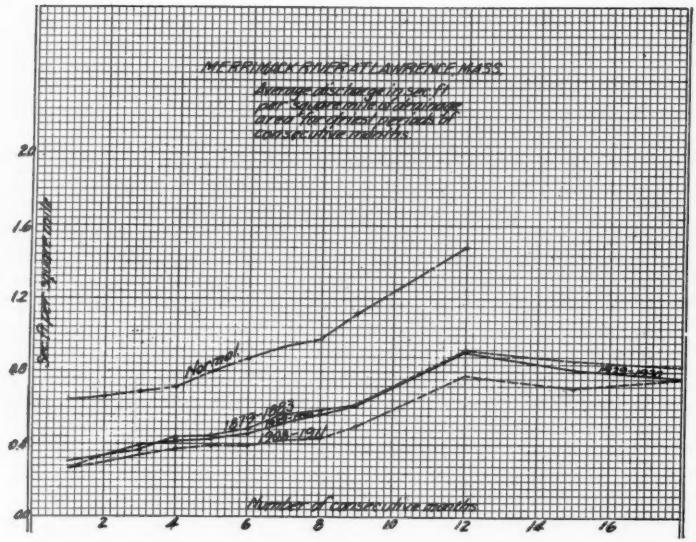
MERRIMACK RIVER AT LAWRENCE, MASS.
Average discharge in sec. ft. per square mile of drainage for the greatest periods of consecutive months.
Number of consecutive months
Normal
1879-1903
1903-1931
1879-1931
2.0
1.6
1.2
0.8
0.4
0.0
2
4
6
8
10
12
14
16

FIG. 3.

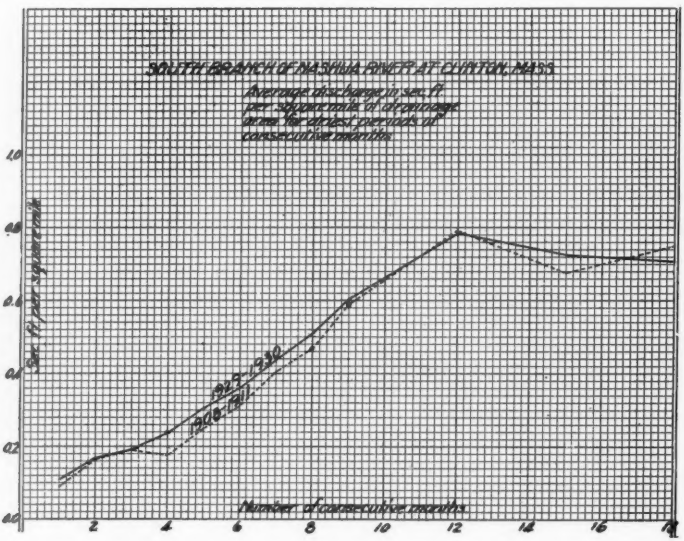
SOUTH BRANCH OF NASHUA RIVER AT CLINTON, MASS.
Average discharge in sec. ft. per square mile of drainage for the greatest periods of consecutive months
Number of consecutive months
1.0
0.8
0.6
0.4
0.2
0.0
2
4
6
8
10
12
14
16

FIG. 4.

TABLE 4.— COMPARISON OF DRY PERIODS BASED ON AVERAGE DISCHARGE.

Consecutive Months.	Pemigewasset. Sec. ft. per sq. Mile.		Ashuelot. Sec.-ft. per sq. Mile.		Conn. (Orford.) Sec.-ft. per sq. Mile.		Conn. (Sunderland.) Sec.-ft. per sq. Mile.	
	1908–1912.	1929–1930.	1908–1912.	1929–1930.	1908–1912.	1929–1930.	1908–1912.	1929–1930.
1	0.345	0.405	0.148	0.167	0.202	0.392	0.229	0.328
2	.367	.461	.163	.217	.222	.417	0.229	.364
3	.427	.562	.183	.233	.259	.465	.238	.404
4	.446	.630	.228	.251	.291	.538	.284	.453
5	.576	.744	.246	.272	.342	.623	.340	.504
6	.675	.820	.270	.292	.367	.610	.358	.534
7	.736	.903	.351	.366	.439	.693	.413	.615
8	.786	1.075	.423	.428	.545	.735	.521	.730
9	.844	1.138	.504	.501	.615	.788	.622	.825
12	1.630	1.816	.798	.846	1.267	1.405	1.079	1.279
15	1.626	1.755	.760	.741	1.160	1.320	.971	1.178

TABLE 5.— YIELD OF DRAINAGE BASINS FOR DRIEST PERIODS OF CONSECUTIVE MONTHS, JUNE, 1929 TO SEPTEMBER, 1930.
(Sec.- ft. per sq. Mile.)

Consecutive Months.	Souhegan.	Swift.	Ware.	Quaboag.	Sip Pond Brook.	Westfield at Knightville.	Westfield at Westfield.
1	0.135	0.239	0.241	0.183	0.127	0.144	0.234
2	.150	.256	.271	.196	.132	.166	.238
3	.156	.317	.295	.255	.139	.176	.274
4	.186	.370	.308	.315	.150	.272	.350
5	.203	.405	.341	.349	.160	.344	.424
6	.221	.434	.376	.390	.193	.407	.445
7	.301	.501	.442	.448	.227	.502	.565
8	.367	.556	.498	.497	.279	.650	.666
9	.472	.611	.571	.548	.339	.776	.758
12	.878	.791	.748	.696	.640	1.146	1.086
15	.753	.712	.709	.623	.595	1.084	1.019

In comparing the current drought on this watershed with that following 1908, we find that for three consecutive months in both periods the yield is practically the same, after which the current drought is less severe except for twelve and eighteen months. This indicates that the drought to date on the Nashua River is just as severe if not more so than the drought following 1908, which has been considered the most severe drought on record.

Table 4 and Figures 5–8 compare the normal yield with the yield for the drought following 1908 and 1929 at four gaging stations. Only on the Ashuelot River is it found that the 1929 drought is as severe as that following 1908. Table 5 and Figure 9 show the yield for various periods of consecutive months for several gaging stations in the area affected by the current drought. Discharge data for the earlier droughts are not available for these gaging stations.

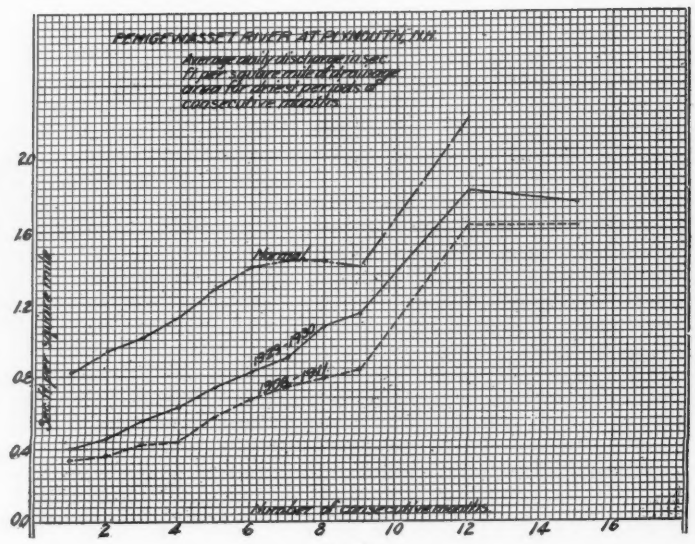
PEMIGEWASSET RIVER AT PLYMOUTH, NH.
Average daily discharge in sec. ft. per square mile of drainage area for the best periods of consecutive months.
Sec. ft. per square mile
Normal
1929
1930
1911
Number of consecutive months
0.0
0.4
0.8
1.2
1.6
2.0
2
4
6
8
10
12
14
16

FIG. 5.

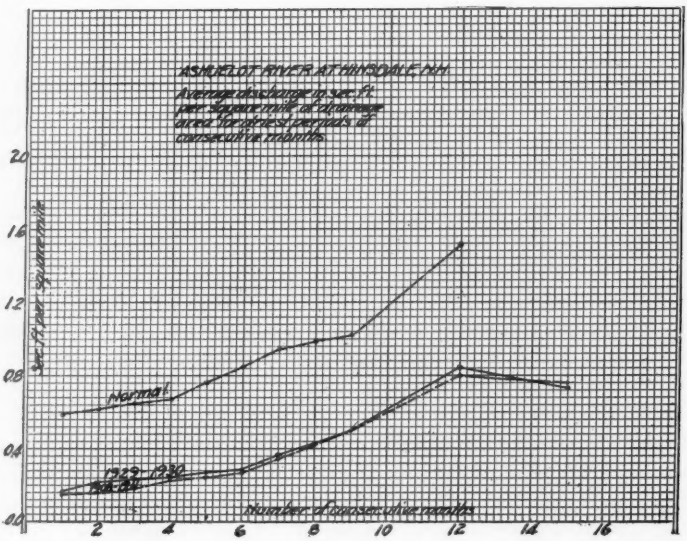
ASHUELOT RIVER AT HINSDALE, NH
Average discharge in sec. ft. per square mile of drainage area for the best periods of consecutive months.
Sec. ft. per square mile
Normal
1929
1930
Number of consecutive months
0.0
0.4
0.8
1.2
1.6
2.0
2
4
6
8
10
12
14
16

FIG. 6.

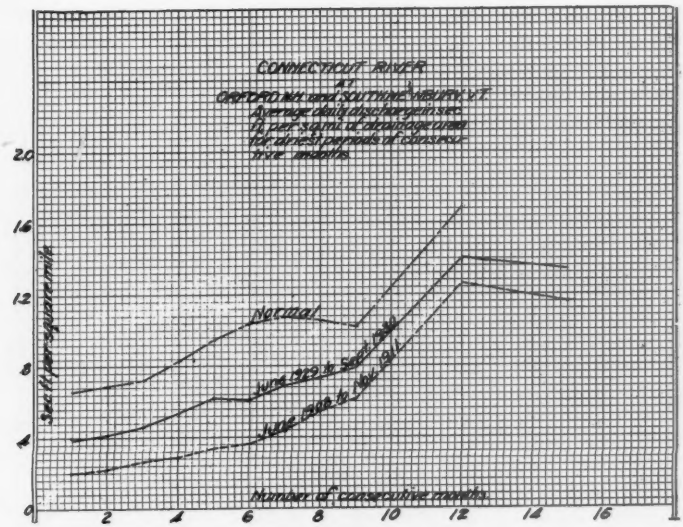
CONNECTICUT RIVER
AT
CHELSEA, VT. AND SUTTON, MILTON, VT.
Average daily discharge in sec.
ft. per sq. mi. of drainage area
for different periods of consecutive
three months.
Sec. ft. per square mile
Normal
June 1929 to Sept. 1930
June 1930 to Sept. 1931
Number of consecutive months
0
4
8
12
16
20
2
4
6
8
10
12
14
16

FIG. 7.

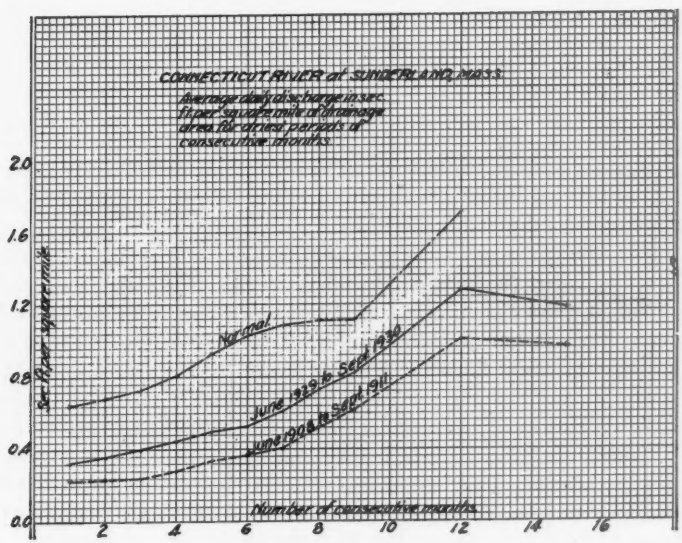
CONNECTICUT RIVER at SUNDERLAND, MASS.
Average daily discharge in sec.
ft. per square mile of drainage
area for different periods of
consecutive months.
Sec. ft. per square mile
Normal
June 1929 to Sept. 1930
June 1930 to Sept. 1931
Number of consecutive months
0.0
0.4
0.8
1.2
1.6
2.0
2
4
6
8
10
12
14
16

FIG. 8.

The average yearly yield over a 14-year period beginning in 1916 on Sip Pond Brook, a tributary of Millers River near Winchendon, Mass., was 1.65 sec.-ft. per sq. mile. In the year ending September 30, 1930 this yield dropped to .686 sec.-ft. per sq. mile, which is but 42 per cent. of the normal, and the lowest year in the period of record.

On the Quaboag River at West Brimfield, the average yield for the

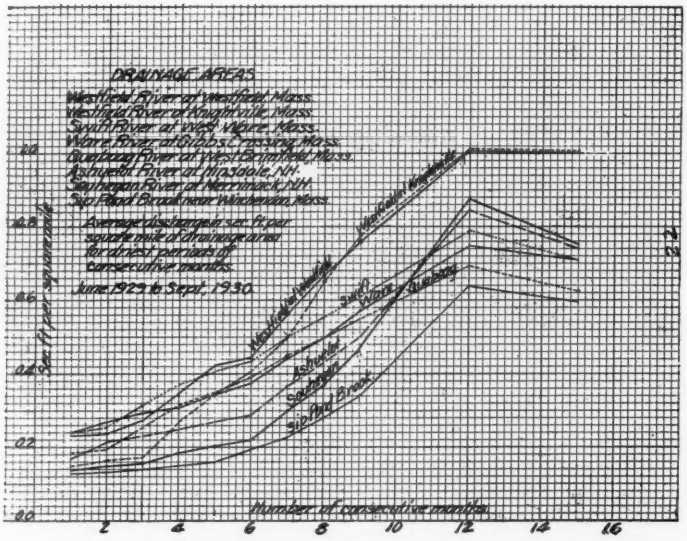
DRAINAGE AREAS
Westfield River at Westfield, Mass.
Westfield River at Knightville, Mass.
Swift River at West Ware, Mass.
Ware River at Gibbs Crossing, Mass.
Quaboag River at West Brimfield, Mass.
Ashuelot River at Hinsdale, N.H.
Saltonstall River at Hartwick, N.H.
Sip Pond Brook near Winchendon, Mass.
Average discharge in sec. ft. per square mile of drainage area for driest periods of consecutive months.
June 1923 to Sept. 1930.
Number of consecutive months
2
4
6
8
10
12
14
16

FIG. 9.

year ending September 30, 1930 was .693 sec.-ft. per sq. mile, the lowest in a period of 18 years dating from 1912 and but 45 per cent. of the normal yield of 1.55 sec.-ft. per sq. mile.

Practically the same result was obtained on the Swift River at West Ware, Mass., where the yield of .796 sec.-ft. per sq. mile for the year ending September 30, 1930 was recorded, which was but 49 per cent. of the average yield of 1.63 sec.-ft. per sq. mile for the 18 years of record dating from 1912.

On the Westfield River at Knightville with a 21-year record dating from 1909, we find that the year ending September 30, 1911 gave the smallest yield, which was 55 per cent. of normal as compared to a yield of 1.31 sec.-ft. per sq. mile for the year ending September 30, 1930, which was 67 per cent. of normal. This last figure is closely checked by the yield of the Westfield River near Westfield, which for the year ending September 30, 1930 was 1.72 sec.-ft. per sq. mile, which was 66 per cent. of normal.

Our discharge data were not entirely complete for the drought following 1909 on the Ashuelot at Hinsdale, N. H., but were complete through the

TABLE 6.—MONTHLY DISCHARGE IN SECOND-FEET PER SQUARE MILE.

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Mean for Year
PEMIGEWASSET RIVER AT PLYMOUTH, N. H. Drainage area, 615 sq. miles.													
1928–29	.872	1.37	.995	1.19	.673	3.98	7.30	6.31	1.40	.763	.405	.517	2.15
1929–30	.833	1.20	1.20	2.28	1.64	3.51	4.98	3.07	3.17	1.00	1.35	.411	2.05
SOUHEGAN RIVER AT MERRIMACK, N. H. Drainage area, 168 sq. miles.													
1928–29	.708	.679	.851	1.52	1.11	5.31	5.45	3.39	.780	.312	.168	.143	1.71
1929–30	.156	.278	.268	.827	1.32	3.17	1.90	1.21	.798	.421	.193	.135	.887
CONNECTICUT RIVER AT SO. NEWBURY, VT. Drainage area, 2 830 sq. miles. Corrected for storage at First and Second Connecticut Lakes.													
1928–29	1.59	1.44	1.06	.993	.502	2.24	5.62	5.16	1.21	.756	.442	.392	1.79
1929–30	.562	.961	.548	1.19	1.03	1.75	4.35	3.67	2.10	1.11	.534	.413	1.52
ASHUELOT RIVER AT HINSDALE, N. H. Drainage area, 440 sq. miles.													
1928–29	.773	.645	.793	1.10	.620	3.77	4.36	3.23	.770	.341	.252	.205	1.41
1929–30	.209	.291	.377	.821	1.04	2.63	1.63	1.14	.745	.430	.332	.159	.816
MERRIMAC RIVER AT LAWRENCE, MASS. Drainage area, 4 452 sq. miles since Mar. 7, 1898. Record furnished by Essex Company.													
1928–29	.977	.979	1.12	1.54	1.49	3.65	3.99	3.42	.973	.583	.400	.365	1.62
1929–30	.410	.502	.497	.878	.965	2.267	2.053	1.179	1.058	.523	.521	.379	.935
SOUTH BRANCH OF NASHUA RIVER NEAR CLINTON, MASS. (Wachusett Drainage Basin). Drainage area, 108.84 sq. miles. Record furnished by Metropolitan District Commission.													
1928–29	.616	.696	1.040	1.856	2.218	4.260	4.445	3.341	.914	.230	.109	.243	1.664
1929–30	.370	.565	.641	1.002	1.341	1.861	1.477	.902	.693	.701	.346	.364	.855
SUDBURY RIVER BASIN NEAR FRAMINGHAM, MASS. Drainage area, 75.2 sq. miles. Record furnished by Metropolitan District Commission.													
1928–29	.814	.912	1.301	2.037	2.337	4.121	4.551	2.775	.353	−.172	−.091	−.039	1.575
1929–30	.084	.393	.546	.965	1.455	2.241	1.533	.626	.069	.036	.128	.280	.696
LAKE COCHITUATE BASIN NEAR COCHITUATE, MASS. Drainage area, 17.58 sq. miles. Record furnished by Metropolitan District Commission.													
1928–29	.941	.875	1.369	2.021	2.084	4.021	4.358	2.314	.372	−.028	−.049	−.032	1.520
1929–30	.150	.320	.713	1.220	1.589	1.870	1.308	.461	.188	.091	.098	.077	.674
CONNECTICUT RIVER AT SUNDERLAND, MASS. Drainage area, 8 000 sq. miles. Since 1924, corrected for storage at Connecticut Lakes, Somerset and Davis Bridge reservoirs.													
1928–29	1.01	1.05	1.01	1.31	.701	3.72	5.61	4.39	1.10	.599	.360	.368	1.78
1929–30	.485	.710	.682	1.54	1.58	2.74	3.06	2.12	1.80	.825	.476	.328	1.36
MILLERS RIVER AT ERVING, MASS. Drainage area, 372 sq. miles.													
1928–29	1.17	1.00	1.01	1.62	1.58	4.33	4.62	3.52	1.20	.594	.285	.230	1.77
1929–30	.241	.390	.392	.858	1.18	1.85	1.70	1.02	.720	.737	.280	.244	.798
SIP POND BROOK NEAR WINCHENDON, MASS. Drainage area, 18.8 sq. miles.													
1928–29	.878	.729	.739	1.20	.713	4.10	5.03	3.44	.824	.363	.181	.152	1.53
1929–30	.127	.138	.202	.424	.644	2.10	1.57	1.04	.739	.654	.381	.208	.686
WARE RIVER AT GIBBS CROSSING, MASS. Drainage area, 199 sq. miles.													
1928–29	.864	.724	.930	1.33	1.27	4.30	4.44	3.16	.839	.348	.255	.287	1.56
1929–30	.342	.474	.548	.889	1.16	1.68	1.47	.799	.719	1.11	.316	.241	.809
SWIFT RIVER AT WEST WARE, MASS. Drainage area, 186 sq. miles.													
1928–29	.866	.812	.909	1.43	1.26	3.83	3.90	3.28	.941	.529	.287	.320	1.53
1929–30	.345	.543	.581	.903	1.05	1.82	1.42	.909	.785	.683	.272	.239	.796
QUABOAG RIVER AT WEST BRIMFIELD, MASS. Drainage area, 150 sq. miles.													
1928–29	.893	.840	.867	1.47	1.32	3.62	4.05	3.56	.847	.445	.327	.331	1.55
1929–30	.361	.392	.592	.840	.953	1.65	1.59	.720	.495	.371	.210	.183	.693
WESTFIELD RIVER AT KNIGHTVILLE, MASS. Drainage area, 162 sq. miles.													
1928–29	.533	.536	.778	1.11	.858	6.42	7.04	3.25	.630	.189	.144	.194	1.81
1929–30	.563	.722	1.07	1.69	1.78	3.07	2.31	1.39	1.74	.778	.357	.260	1.31
WESTFIELD LITTLE RIVER NEAR WESTFIELD, MASS. Drainage area, 48.5 sq. miles.													
1928–29	.454	.540	.889	2.76	1.96	6.08	6.91	2.62	.485	.115	.066	.126	1.92
1929–30	.728	.924	1.46	1.70	1.59	3.22	2.43	1.49	1.66	.476	.210	.200	1.34

NOTE: All previous data for stream gaging stations located in New England, with the exception of the State of Maine, are published in the *Journal of the Boston Society of Civil Engineers*, May, 1930.

calendar year of 1911. For the year ending September 30, 1911, the average yield was .845 sec.-ft. per sq. mile or 59 per cent. of normal, while for the year ending September 30, 1930 the yield was .816 sec.-ft. per sq. mile, or 57 per cent. of normal. This does not indicate conclusively that the yield was less during our current drought than during the former since several months of record were missing between 1911 and 1915. (See Fig. 6.)

On the Souhegan River at Merrimack, N. H., the yield in sec.-ft. per sq. mile for the year ending June 30, 1930, was .875 as compared to .839 for the year ending September 30, 1911, the driest year since the record started in 1909. These values represent percentages of 53 and 51 of normal yield respectively.

These are but some of the highlights taken from the tables of comparative discharge and indicate the relative severity of the most recent drought. They conclusively place it on an equal footing with the most severe drought within the period of available stream-flow data with a possibility that it may take first rank should the current drought continue into next year. Table 6 gives the monthly discharge in second feet per sq. mile for the years ending September 30, 1929 and 1930, at gaging stations, in the area affected by the drought where long-term records are available.

This report should be considered in the light of a progress report as it is yet too early to predict whether or not this drought may continue into the coming year.

Experience Outside New England. Charles A. Mixer writes from Rumford, Me., on the Androscoggin River, that they have experienced no drought this year and states that there was an excess of rainfall during the four months ending August 31, 1930 of 2.50 in. It is also his opinion that we are now in the middle of a ten-year drought with a present accumulative deficiency of about 21 in. Information received from other sections in Maine indicates, however, that in the central part of that state the precipitation was about 70 per cent. of normal, and in eastern Washington County only 60 per cent. of normal, but that the yields from the watersheds have not been noticeably deficient. It is also stated that the 1911 drought was the most severe on record.

Information obtained from Connecticut indicates that the drought started in May, 1929, and that, except for three months, the deficiency accumulated through October, 1930. Rainfall was about 80 per cent. of normal while the stream flow was only 67 per cent. The most serious drought previous to this occurred during the period May, 1912 to February, 1913. During October and November, 1913, the streams of Connecticut reached the minimum on record.

In the state of New York it appears that the drought occurred in two periods — July to October, 1929, and April to October, 1930,— but it is considered there that the drought is not yet broken. Considering the whole period from July, 1929 to November, 1930, their precipitation has been about 85 to 90 per cent. of normal, some sections of the state receiving greater

than normal rainfall a part of the time. River discharge during this period on the Susquehanna River at Conklin, N. Y., was only 64.2 per cent. of normal. The average stream flow for the state appears to have been about 75 per cent. of normal. In the eastern, southern and western parts of New York State, the 1930 drought has been the most serious for the period of record extending from 1908 to 1930.

In New Jersey the drought has not been so severe, rainfall averaging about 80 per cent. of normal, while yield was approximately 70 per cent. of normal. This drought was practically the same as the 1925 drought.

Farther down the coast in North Carolina the drought did not make itself felt until December, 1929, but has been quite severe for a relatively short period, the precipitation in different drainage areas amounting to but 15 to 30 per cent. of normal, with corresponding yields in these watersheds of from 20 to 35 per cent. This drought was not so severe nor so protracted as the one of 1925–1926.

In West Virginia the drought started in December, 1929, and rainfall has been deficient each month since, amounting at the end of October, 1930, to a deficiency of over 17 in. The percentage of normal rainfall received during this period is 58. The period from August to November, 1930, may show a yield as low as 10 per cent. of normal on some of the streams, indicating the most severe drought on record. Our district engineer located in that state advises that if contemplating a visit to that district one's own drinking water should be brought along as water there is scarce and hardly fit for drinking.

Florida was not in the drought section.

In the central states the drought did not begin to show its effect until April, 1930. Since that time in Ohio the rainfall has been 61 per cent. of normal while the stream flow is about 40 per cent. This is the most severe drought on record.

Approximately the same conditions are indicated from information received from Illinois, Indiana, Kansas, Missouri, Arkansas, Minnesota and other sections of the central portion of the United States.

In the state of Oregon stream flow was deficient in every month of the climatic year, except in December and February, and during the period April to September, 1930, the rainfall was 89 per cent. of normal, resulting in a run-off, which in some cases was but 36 per cent. of normal. As far as run-off is concerned, this is the most severe drought on record.

Undoubtedly other records of yield have been secured by private concerns or officials connected with the supply of water. If they are available and are secured with proper accuracy they would be valuable in a study such as this, especially if they are for a long period and could be published with the rest of stream-flow data in our yearly publications.

Discussion.

Arthur L. Shaw.* Reports from various quarters indicate a wide divergence in the severity of the effect of the current dry period upon different water supplies of substantially equal adequacy under ordinary circumstances, and it may be of interest to consider one of the reasons why this can be so.

The effect of a so-called dry year upon a water supply depends not only upon the distribution of run-off throughout the whole year, but also upon

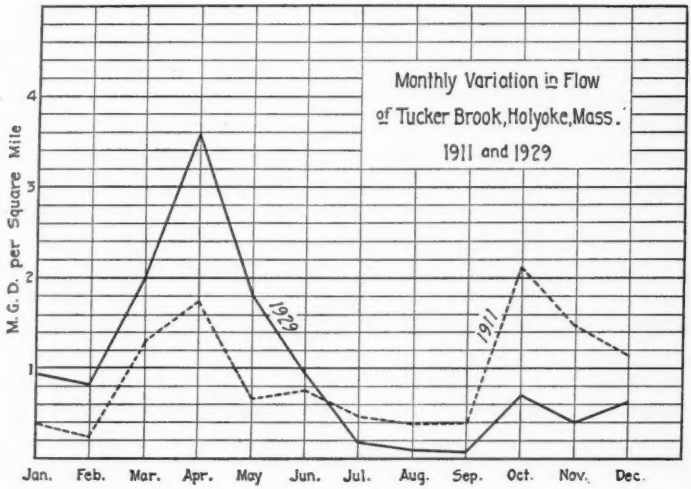

Fig. 10.

the type of water-supply system and the basis upon which it has been designed and developed.

The records of flow in Tucker Brook, as observed by the Holyoke Water Department, for the dry years 1911 and 1929 will serve to illustrate two types of dry year. The monthly average rates of flow in this brook are shown diagrammatically in Figure 10. Both years have an annual average run-off of close to 1 m.g.d. per sq. mile and still each would affect a given water-supply development in quite a different manner. The year 1929, with fairly high spring flows, experienced unusually low summer flows, extending into the fall months. The year 1911, on the other hand, was deficient during the spring, and low flows commenced earlier in the season than usual, but at their lowest during July, August and September, were very much higher than in 1929.

*Of Metcalf & Eddy, Engineers, Boston, Mass.

Water-supply systems well provided with impounding reservoirs presumably started 1929 with full storage basins and were, therefore, for the most part, not seriously inconvenienced by the summer drought. On the other hand, those systems designed to depend upon natural flows, or those with deficient storage capacity suffered from the extremely low flows of the summer and fall months of 1929.

In contrast with 1929 a dry year like 1911 would be more unfavorable for stored supplies, due to the failure of reservoirs to fill in the spring; whereas the natural-flow systems would suffer much less than in 1929 since the summer flows were sustained at a much higher rate.

These facts will explain, at least in part, why the reports from different communities differ so widely as to the severity of a given dry year. What is a very bad year for some types of development may not be difficult for others, and yet the tables may be turned when the next dry year, with its different characteristics, comes along.

Often the characteristics mentioned will not be clearly evident, especially when the drought extends over more than one year, successive dry years being of different types. Thus a succession of dry years is likely to result in a leveling of all supplies to a somewhat uniform state of distress, except in those communities which have a large margin of safety in their water-supply developments.

ARTHUR D. WESTON.* The rainfall throughout the state as a whole from June to December, 1929, inclusive, as shown by the average rainfall at Amherst, Boston, Lowell, Leominster, Reading, New Bedford, Taunton and Northbridge was 6.50 in. below the normal while during the period of January to September, 1930, inclusive, the rainfall was 10.15 in. below the normal, making the total deficiency from June, 1929 to September, 1930, inclusive, 16.65 in. The rainfall for October, 1930 was 4.30 in. and that for November, 3.89 in.; October was 17 per cent. above the normal while November was normal. In December, there was a deficiency of 1.26 in., making the total deficiency in rainfall from June, 1929 to December, 1930, inclusive, throughout the state 17.28 in.

On the Wachusett Drainage Area the deficiency in rainfall from June to December, 1929, inclusive, as recorded at four stations was 8.86 in., while the deficiency from January to December, 1930, inclusive, was 9.85 in. Throughout the entire period from June, 1929 to December, 1930, inclusive, the deficiency was 18.71 in.

As a result of this deficiency in rainfall, there has been a deficiency in run-off from most of the watersheds throughout the state. From the Wachusett Watershed for the 19 months from June, 1929 to December, 1930, inclusive, the run-off was equivalent to but 15.373 in. of rainfall collected or a deficiency of 15.169 in. This is the lowest run-off recorded on this watershed for a like period since the records were first begun. Table 7 shows the rainfall collected on this drainage area for different

*Chief Engineer, Massachusetts State Department of Health, Boston, Mass.

periods beginning with June, 1897 in order of dryness, and Figure 11 presents the accumulative deficiency in rainfall since June 1, 1929 as recorded at certain rainfall stations throughout the state.

In consequence of the abnormally low rainfall, many water supplies in the state were depleted during the year 1929, and it was necessary for the

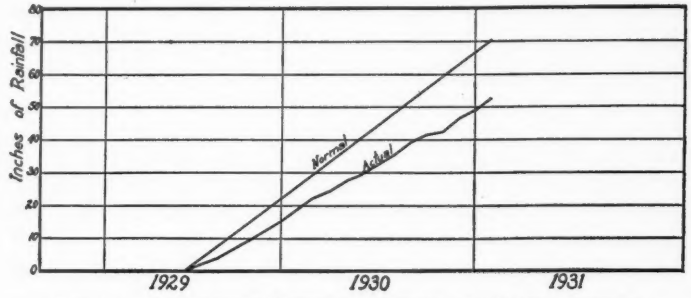

FIG. 11.— CUMULATIVE MONTHLY RAINFALL AT 8 MASSACHUSETTS STATIONS, JUNE, 1929 TO SEPTEMBER, 1930, INCLUSIVE.

following municipalities to make arrangements relative to new or emergency sources of water supply:

Brookfield — Drove tubular wells.
Millbury Water Co.— Took water from Worcester.
Northfield Water Co.— Took water from Warwick Brook.
Pittsfield — Took water from Onota Lake.
Shelburne Falls Fire District—Took water from Deerfield River and enlarged reservoir.
Worcester — Took water from Wachusett Reservoir.

The continuation of the drought into the year 1930 resulted in a larger number of supplies being depleted. The supplies of the following municipalities were affected:

Brookfield — Obtained additional supply from new dug wells.
Gloucester — Obtained water in part from the town of Manchester and constructed an additional storage reservoir.
Grafton — Obtained water from new tubular well and supplied water in part to the town of Millbury.
Holliston — Drove additional tubular wells.
Millbury — Obtained water in part from Grafton Water Company.
Needham — Obtained additional supply from tubular wells near Charles River.
Reading — Obtained additional supply from tubular wells.
Wakefield — Obtained additional supply in part from Metropolitan Water Works through town of Stoneham and in part from tubular wells.
Worcester — Obtained additional supply by pumping from Wachusett Reservoir.
Pittsfield — Obtained additional supply from Onota Lake.

TABLE 7.— PERIODS OF MINIMUM RAINFALL COLLECTED ON WACHUSETT WATERSHED, 1897 TO 1930, INCLUSIVE IN ORDER OF DRYNESS.

Period.	Number of Months.	Years.	Inches Collected.
June to March.	10	1929–1930	8.179
		1910–1911	8.320
		1924–1925	10.394
June to April.	11	1929–1930	9.822
		1910–1911	10.724
		1914–1915	12.308
June to May.	12	1929–1930	10.861
		1910–1911	11.564
		1914–1915	13.119
June to June.	13	1929–1930	11.634
		1910–1911	12.152
		1914–1915	13.512
June to July.	14	1910–1911	12.253
		1929–1930	12.442
		1924–1925	15.239
June to August.	15	1910–1911	12.588
		1929–1930	12.668
		1924–1925	15.586
June to September.	16	1929–1930	12.901
		1910–1911	12.901
		1924–1925	16.182
June to October.	17	1929–1930	13.555
		1910–1911	14.181
		1924–1925	16.961
June to November.	18	1929–1930	14.365
		1910–1911	15.967
		1924–1925	18.339
June to December.	19	1929–1930	15.026
		1910–1911	17.870
		1924–1925	21.236

In addition, the towns of Rutland and Holden and the Hingham Water Company in Hingham were obliged to extend their intake works to permit drawing water from their sources of supply at a lower level. The sources of supply of the Metropolitan Water District have also been considerably depleted, while those of the city of Cambridge have been reduced to such a low level that the city has investigated additional water supplies.

EFFECTS OF THE 1929–1930 DROUGHT UPON THE WATER SUPPLY OF ROCKPORT, MASS.

BY G. C. HOUSER.*

[*Received December 12, 1930.*]

The town of Rockport, on Cape Ann, is one of the popular summer resorts of the Massachusetts coast. For this reason the number of people living in the town during the summer months is about three times the official population. The permanent population of the town, which has been decreasing since 1895, is given by the latest Federal census as 3 630. The summer population has been increasing, and, while there are no official figures available, it is estimated that in 1930 it was between 10 000 and 12 000 persons.

Rockport's principal industry, at least in the past, has been granite quarrying. During recent years, however, there has been a depression in the granite business in Rockport. Several of the quarries have been abandoned and some of them are now filled with water.

Sources of Water Supply. The principal source of water supply for the town is Cape Pond, a natural body of water, located in the southerly part of the town. The watershed consists largely of woodland, covering an area of approximately 0.4 sq. mile. At ordinary high water the pond itself has an area of 43 acres, with an average depth in the neighborhood of 12 ft., a maximum depth of 23 ft., and a total storage capacity of 183 m.g. Due to the fact that water is drawn out of the pond at a depth of 15 ft. below high-water mark, the available capacity of the pond when filled is only 165 m.g. The pond water is chlorinated before being pumped to a standpipe for distribution to the mains.

For many years the entire supply was obtained from Cape Pond, which was so seriously taxed at times that the water commissioners found it desirable to restrict the use of water, particularly for sprinkling. In 1926 an auxiliary water supply was developed from four tubular wells in the valley of Mill Brook, in the central part of the town. These wells yielded about 80 000 g.p.d. on an average, when placed in daily service for a month at a time. The well supply was intended to be used for reducing the draft on Cape Pond in emergencies, and it was not anticipated that it would be necessary to make use of it for more than two months of the year.

Yield of Cape Pond Compared with Water Consumption. The water level of Cape Pond is not controlled by the town, and water flows to waste through the outlet brook whenever the water in the pond is higher than the bed of the brook. A study of the gage elevations of the pond, and the corresponding depth of water at the outlet, shows that there was some overflow

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during each of the seven years from 1923 to 1929, inclusive, although the quantity was insignificant in 1924. On the other hand, during 1930 the level of the pond has at all times been at least 1.75 ft. below the overflow. The data in regard to overflows are insufficient to enable a satisfactory estimate to be made of the quantity of water wasted at the outlet. If this portion of the yield is neglected entirely, there remains for consideration that portion which is either pumped to the distribution system or left in storage in the pond, or, in other words, the available yield.

During the past eight years, the available yield of Cape Pond has averaged 8.6 m.g. per month, while the monthly quantity of water pumped from the pond has averaged 9.1 m.g. (see Table 1). Stated in another way, there has been an excess of pumpage over available yield during this eight-year period amounting to 51 m.g.

This deficit in the quantity of water in storage was brought about by the recent drought, which began to be felt at Rockport early in May, 1929, and lasted until the latter part of October, 1930. During this period of eighteen months, the available yield of Cape Pond averaged only 5.7 m.g. per month, while the monthly quantity of water pumped from the pond averaged 10.6 m.g. (see Table 2). In other words, during this period of $1\frac{1}{2}$ years there was an excess of pumpage over available yield amounting to 88 m.g.

Quantity of Water in Storage. Of course, the effect of the drought upon Cape Pond was evident to the casual observer, for the available quantity of water in storage on November 1, 1930, was only 47 per cent. of the quantity on May 1, 1929. On the latter date the pond was high, having an available storage of 167 m.g. (see Table 3). Falling steadily throughout the summer and autumn, on December 1, 1929, the storage stood at 95 m.g. The run-off during the winter and spring increased the storage to 129 m.g. on June 1, 1930, but by November 1 it had dropped to 79 m.g., the level of the pond on this date being 6.6 ft. below ordinary high water.

TABLE 1.— COMPARISON OF AVAILABLE YIELD OF CAPE POND WITH QUANTITY OF WATER PUMPED FROM POND, BY YEARS, 1923–1930.

Year.	Availabe Yield of Cape Pond.* (Mill. Gal.)	Quantity Pumped from Pond. (Mill. Gal.)
1923	107	134
1924	121	104
1925	119	107
1926	94	100
1927	122	95
1928	81	96
1929	70	116
1930†	107	120
Total	821	872
Annual Average	103	109
Monthly Average	8.6	9.1

*Neglecting water wasted at pond outlet.
†Estimated from figures for first 11 months.

TABLE 2.— COMPARISON OF AVAILABLE YIELD OF CAPE POND WITH QUANTITY OF WATER PUMPED FROM POND, BY MONTHS, MAY, 1929 TO OCTOBER, 1930.

Month.	Available Yield of Cape Pond.* (Mill. Gal.)	Quantity Pumped from Pond. (Mill. Gal.)
May, 1929	5	9
Jun.	1	13
Jul.	0	17
Aug.	4	14
Sep.	−5	11
Oct.	−1	9
Nov.	5	8
Dec.	16	8
Jan., 1930	22	7
Feb.	8	8
Mar.	18	8
Apr.	8	7
May	8	8
Jun.	3	11
Jul.	1	14
Aug.	1	15
Sep.	0	14
Oct.	9	10
Total	103	191
Monthly Average	5.7	10.6

*Neglecting water wasted at pond outlet during May and June, 1929; no overflow in subsequent months.

TABLE 3.— AVAILABLE QUANTITY OF WATER STORED IN CAPE POND ON THE FIRST DAY OF EACH MONTH, 1929 AND 1930.

Month.	Available Storage. (Mill. Gal.)	
	1929.	1930.
Jan.	148	103
Feb.	158	118
Mar.	162	118
Apr.	165	128
May	167	129
Jun.	163	129
Jul.	151	121
Aug.	134	108
Sep.	124	94
Oct.	108	80
Nov.	98	79
Dec.	95	91

Auxiliary Well Supply. In the summer of 1929, the four tubular wells in the Mill Brook valley were used for about two months, in order to reduce the draft on Cape Pond. During this time the total quantity of water

pumped from the wells, based on pump displacement, was approximately 2 m.g. The draft on the wells was so heavy that one of the wells went out of service, and it was found advisable to discontinue pumping, in order that the other three wells might not be drawn upon too heavily.

In 1930, the pumping of water from the wells was begun in the middle of April. By the first of June it had become evident that steps should be taken to augment the water supply, in order to reduce the possibility of a water shortage. For the purpose of increasing the yield of the well supply, four new wells were driven in June and connected with the pump serving the four old wells. Subsequent to the completion of this work, all eight wells were drawn upon until the early part of November, the total quantity of water pumped from the wells during the year being approximately 13 m.g. The use of this comparatively large quantity of well water was of great value in preventing Cape Pond from being drawn down to a critically low level.

Emergency Supply. During the past summer the town of Rockport had under consideration a plan for obtaining additional water in case of an extreme emergency. This plan involves drawing water from one of the abandoned granite quarries in the northern part of the town. Although the quantity of water contained in the quarry pit is not accurately known, it is estimated that an amount equivalent to at least six months' consumption is available. In order to use the water, it would be necessary to set up pumping equipment and lay a pipe from the pit to the existing street mains, a distance of approximately 1 500 ft. The emergency line would probably consist of cast-iron pipe with composition joints, laid on the surface of the ground. In addition, chlorinating apparatus would have to be provided, in order to make the water safe in case of possible contamination.

Fortunately, the recent drought was not severe enough to make the adoption of such an emergency supply necessary. If 1931 should chance to be another very dry year, it is likely that Rockport will follow this scheme of drawing water from an abandoned quarry.

Conclusion.

Summarizing, it has been shown that the drought of 1929–1930 reduced the quantity of water in storage in Cape Pond to 47 per cent. of the amount available just before the drought began, in spite of the use of a tubular-well supply to reduce the consumption of Cape Pond water. On the other hand, the drought was not severe enough to compel the town to draw an emergency water supply from an abandoned granite quarry, although such a measure was under consideration.

PRACTICAL POINTS IN THE OPERATION OF TOWN WATER-WORKS PUMPS.

BY CHARLES E. GREENE.*

[*Read March 11, 1931.*]

The writer expects to discuss some of the practical points of pump operation and control that have recently come under his observation. Most of these have to do with electric motor-driven centrifugal pumps.

PRIMING CENTRIFUGAL PUMPS.

It is universally accepted that a centrifugal pump ought to be placed so that it is always primed, *i. e.*, it must be below the water level from which it is pumping. This can always be done by sufficient expenditure of time for study and money for construction. In spite of this, almost every installation results in a compromise between the cost of locating the pump so that it will always be primed and the risks that result from placing it higher.

Vacuum Priming. On account of these compromises it has become so common to apply vacuum priming to centrifugal pumps that it can almost be regarded as standard practice. The custom is to install some sort of closed system in which a vacuum can be produced when needed or is maintained automatically. The suction piping of the pumps is connected to this vacuum system with the result that the suction water-level is raised sufficiently to prime the pumps.

The designer of such a layout makes the station operators face two risks: (1) that of not being able to start pumping because of some fault in the priming system or because of inability to operate the necessary vacuum pump; (2) that of pumps operating on this basis losing their priming while running, without the conditions being observed promptly enough to prevent damage.

These two risks are so commonly faced nowadays that prevention of them has become a feature of regular pumping procedure rather than a hazard of operation.

Dependable Priming Pumps. We take care of the first risk by providing duplicate vacuum pumps wherever continuity of operation is at all important. If the pumps have an independent means of power for emergency use, such as a gasoline-engine drive in a station normally operating on electric power, the same provision is made for the priming units. In fact, in most modern pumping stations it is more important to provide an alternative drive for the vacuum pumps than for the main pumps, because the

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main pump-motors are of such size that they operate on the primary electric voltage, whereas the vacuum pumps are so small that they must be operated on low voltage. This low voltage is usually obtained from the high-potential supply by station service-transformers and passes through several sets of fuses or other forms of electric protection that make the power supply to the priming unit considerably less reliable than the power supply to the main motors.

Making Air Removal Continuous. The protection against the second set of risks is more complicated. In the old days we made a pump-suction pipe as tight as we could. We had as few connections or chances for air leakage into it as we could. We avoided air traps in it very carefully. Then, when we had filled the pump-suction line and the pump casing (usually by a hand-controlled priming means) and had begun to pump, the system took care of itself.

Since automatic operation of pumps has come into more extended use and especially since centrifugal pumps have been installed with more and more suction lift, we find these pumps with their suction pipes interconnected.

The whole suction system has numerous attachments, gages and vacuum tanks that increase the difficulty of keeping the suction system air-tight and air-free.

Many of these operating problems of the automatically-primed centrifugal pump are actually the same as the problems of reciprocating pumps working under the same conditions. This is only simplified for the reciprocating pump to the extent that the reciprocating pump is its own vacuum pump, and consequently the vacuum is always available when the pump is able to run. It still has to handle the air removed from the water by the vacuum and sometimes is considerably bothered by the resulting mixture of air and water.

Hand-controlled Priming. The pumps at the Dayton Water Works, Ottawa Street Station, were equipped with vacuum-priming piping. These pipes were connected to a loop piping-system in which a vacuum was always maintained, but the vacuum was applied to the pumps by hand-operated valves and only at starting. This has been generally satisfactory except that the operators have some trouble in being sure exactly when the pump casings become filled with water.

The air and water passes through a riser with a sight glass on it. The velocity of the water and air in the piping and the appearance of the water in the glass are such that the operator is likely to keep the priming valve open long enough to fill the whole vacuum system with water instead of merely priming the pump.

Pressure-controlled Priming. A later development introduced at Concord, Mass., and at Allentown, Pa., has been to make the suction pipe of each of the pumps include an air chamber. The vacuum is automatically maintained between certain levels in these suction chambers by vacuum

switches which start and stop one or more of the vacuum pumps, as necessary, to maintain the level. This system works well if the suction levels do not vary too much. The vacuum switch maintains the artificial level a predetermined number of feet above the natural suction level. If the natural suction level fluctuates, the artificial level fluctuates also. Furthermore, the vacuum switch has to have some difference in pressure between its "on"

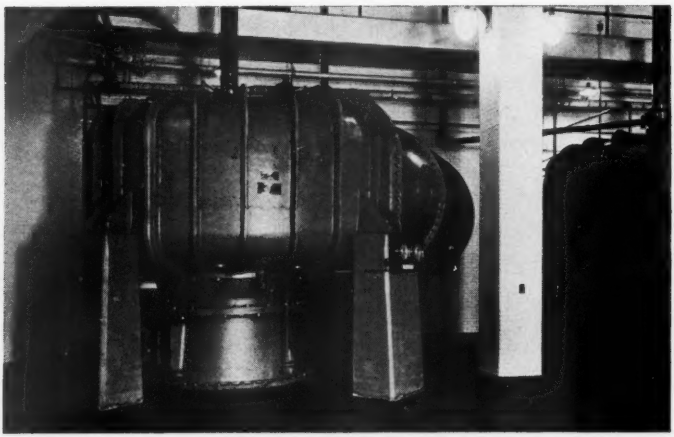

FIG. 1.— CONNORS CREEK PUMPING STATION (for Storm Water), DETROIT, MICH.

500 c.f.s. capacity per pump at 27.5 ft. head, 200 r.p.m. Use of valves avoided by discharging through a siphon. Priming accomplished by raising water level. By reversing pipe connections water can be depressed for easier starting. Vacuum breakers required to prevent back flow.

Pumps, Suction Pipes and Discharge Nozzles.

point and its "off" point. The lowest vacuum in the vacuum system may occur when the natural suction level is also at its lowest point, and the highest vacuum in the vacuum system may occur when the natural suction level is also highest. Hence, the variation in levels is several feet greater than the variation in actual suction-well level. This variation with any given switch setting is usually uncomfortable or difficult to take care of, if the natural suction-level varies. It requires either great height in the station or some adjustment of vacuum-switch setting.

Float-controlled Priming. We are now using a further improvement of the same system where the level in the vacuum system is controlled by float switches instead of by vacuum switches. Obviously, whatever the natural suction-level may be, the float switch will keep the vacuum pumps running as much as necessary to keep the artificial level up to the float-switch setting, whether this means 5-in. vacuum or 15-in. vacuum.

The method of obtaining the air chamber in the pump-suction system and the method of connecting it to the pump-suction inlet is being worked out by different designers to secure the various results called for by local conditions.

Sometimes air removal is the important feature; at other times, economy or reliability are sought.

Continuous Priming. This control takes care of the air that accumulates whether the unit is stopped or running. Consequently the unit is always primed, or "ready to go", if there is power available for the priming-pump motors.

By keeping the pumps always primed, whether running or not, the vacuum pumps can be smaller in capacity than if the vacuum is reduced when the pumps are not running.

It is also necessary to connect the pump casing to the vacuum system so that it can be emptied of air and filled with water before starting the pump. This is done by one or more pipes between the pump casing and the vacuum system. The level maintained by the floats is selected to just fill the casing of the pump. When the pump starts, this pipe comes under pressure and would waste water back to the suction if the valve were not closed. Where the starting and stopping is infrequent and under manual control, this valve is just a gate valve; where the starting and stopping is automatic, this valve is a float valve or magnet valve or other automatically-closed valve.

Protection against Priming Failure. So far as I know, nobody is using any protective features to prevent the pump from operating unprimed. Great precautions are taken to make the priming sure and effective at all times, but complete dependence is put on the operator or on periodic inspection to be sure the desired result is actually obtained.

The ability to prime a centrifugal pump automatically and to keep it primed under all conditions is a big step in the direction of decreasing the number of attendants. Centrifugal pumps can start and stop so handily that the development of automatic priming will undoubtedly bring about the use of smaller, more widely distributed pumping stations, operating without continuous attendance.

Development of Automatic Electric Features.

Considering the other factor in the development of pump operation, the status of electric control and protection, we find that in most systems the combination of actual conditions with the prevailing regard for electric control results in providing automatic electric protection; automatic recording of pressures, rate of flow and power consumption; and automatic control during starting and stopping, but the actual determination of when to start and when to stop is still being left to the station attendant as a general practice.

Automatic Protection. The automatic devices previously mentioned can all react as quickly as necessary to the unseen changes in electrical conditions and load conditions. Many changes in conditions in an electrically-operated station can occur so quickly that an operator could not anticipate them, and could not act quickly enough if he happened to see them occur. Therefore, it is necessary to take the protection against all such troubles out of the operator's hands.

FIG. 2.— CONNORS CREEK PUMPING STATION (for Storm Water), DETROIT, MICH.
Motors, Vacuum Priming Units and Vacuum Breakers.

Automatic Records. Then we install devices to record on a permanent chart the important conditions such as suction level, reservoir level, system pressure and station load. These records let the operator see what is happening and fix responsibility for what has happened. They protect the good operator and help the inexperienced operator to improve.

Automatic Starting. We also install automatic equipment to take care of the various steps connected with starting and stopping electrically-driven units. One reason for this is that the electric protective devices can be set to protect the electric equipment better if the apparatus is automatically started and stopped than if it is started and stopped by hand. Another reason is that electric motors are subject to as much trouble when being started as while in regular running operation. It makes a lot of difference to a motor whether it is started in 4, 7 or 15 seconds. It makes a lot of difference to a synchronous motor whether the field is applied too early or too late or at just the right time. Automatic starters that know how to do these things better than the best operators are available for all types of motors.

We believe the additional investment in automatic starting-equipment is well worth while.

Type of Attendant Determined by Type of Control. The result of this combination is to permit using for station attendants the water-works employee who has grown up with the local water-supply system instead of an operator more skilled in handling electric equipment. Without automatic starting, the operator has to be specially trained to handle the motors properly. With full automatic control, a skilled electrician is often needed to keep the completely automatic system operating perfectly from the electrical point of view.

The water-works type of employee looks at pumping-station performance from a water-supply system point of view. He knows what pressure is needed at various times of the day and knows what can and should be done in case of emergencies, both inside and outside the station.

Typical Control. The development of this idea is shown in the Montclair, N. J., pumping station recently put in commission. Here we have full automatic protection against electrical failure and short circuits both on the incoming lines and on the feeders to each motor. We protect automatically the individual motors against undervoltage, and small sustained overloads during both starting and running and indirectly against a single-phase condition. We give the operator an alarm if a single-phase or undervoltage condition occurs when no pumps are running. To start a pump, the operator merely closes one switch by hand. The rest of the starting operation is controlled automatically.

The operator has the usual indicating pressure-gages and indicating electrical instruments. In addition, he has a chart record of rate of flow, suction pressure, system pressure and electrical input to his pumps.

Pumping into a Closed System. The Montclair system has an interesting feature in that the main centrifugal pumps discharge into a system with no storage on it. The town lies along the side of a mountain with water consumers at such a range of elevations that the water-distribution system is divided into three zones. There is a 2.5-m.g. storage tank on the highest level. If no pumps are running, the middle-pressure zone is fed by pressure-reducing valves from the high-pressure zone, and the low-pressure zone including the town of Glen Ridge is fed in turn by pressure-reducing valves between the middle-pressure zone and the low-pressure zone. The normal method of supply is to pump out of the Wanaque conduit into one end of the low-pressure zone. Another set of pumps near the center of the town pumps from the low-pressure zone distribution-system into the high-pressure zone and storage tank.

Pressure Variation. There is a 30-lb. range in pressures at the pumping station which feeds the low-pressure zone between the maximum allowable pressure in the low zone and the point at which the reducing valves open from the middle zone to the low zone. The centrifugal pumps deliver from 100 per cent. of rated capacity at the high head to 150 per cent. of rated

capacity at the low head, without having their efficiency drop to a wasteful point. The pumps were selected of such capacities that any usual demand for water could be met by one or more pumps operating within the allowable pressure range. The success of the system depended on the frequency, rapidity, and amount of change in rate of consumption. The pumping load has turned out to be a rather "soft" load, *i. e.*, the fluctuations are always so gradual that the operator can change the rate of pumping quickly enough

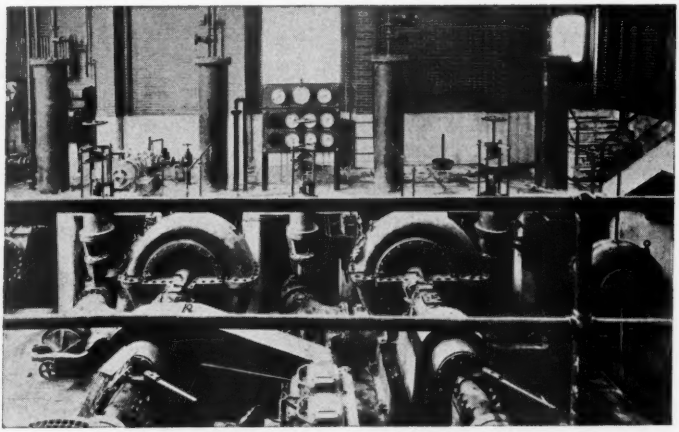

FIG. 3.— SEWAGE PUMPING STATION, ALLENTOWN, PA.

(Automatic Pumping with Vacuum Priming.)

Pump room showing main pumps, priming pumps, risers on suction pipes for vacuum priming, hydraulic operated check valves under solenoid control, and auxiliary.

to meet any requirement. In fact the most serious problem of regulation presented by the system results from the operation of the other station, the relift station. As stated before, there is a group of pumps which lift the water from the low-pressure Glen Ridge zone into the high-pressure zone to fill the tank and to provide for high- and middle-pressure consumption. When this pump starts, it immediately increases the apparent consumption in the low-pressure zone, and the pressure drops accordingly.

However, this is taken care of by telephone communication from one station to the other just before a pump is to be started or stopped or have its rate of pumping changed. With the operator at the other station standing by and watching the gages, he can adjust his rate of pumping so quickly that the disturbance does not exceed the stated allowance.

There was a form of protection already built into this system against the failure of the operators to coördinate. If the relift pump is started without the pumping rate being properly adjusted at the other station, it will have the effect of reducing the pressure in the low-pressure zone by the

sudden increase in consumption in that zone. This opens the pressure-reducing valves between the middle and low zones, which in turn opens the valves between the high and middle zones so that the water which the relift pump is handling would be merely recirculated until the supply of water to the low zone at the other station is built up to include the amount the relift pump is handling plus the consumption in the low zone.

The actual disturbance to the system as shown by the recording pressure-gages at several points is very small. Chart after chart will show no sudden fluctuations of pressure. Those that do show fluctuations seldom exceed the allowable range. This is partly due to the excellent coördination between the operators at the two stations and partly due to the simplicity of control at the new station.

The system has operated for several months now with no inconvenience to the consumer or failure of supply. The operator is able to adjust the pump rates to accommodate variations in demand with ease and accuracy in spite of the unusual characteristics of the system.

Control When Stopping a Centrifugal Pump. This leads to the last point of discussion, the control of electric pumps when stopping. Operators are familiar with the water hammer or surges that can occur when pumps are not stopped carefully. Probably all water-works men would recommend as the best practice that the discharge valve be closed slowly in order to gradually reduce the velocity in the line before stopping a pump. When the valve is finally closed, the power may be cut off from the pump.

As soon as we use electric power for pumping, however, we are faced with the possibility of the power being cut off instantly and without warning. This may happen from a power failure or by the regular or irregular operation of some protective device. This seems to be an inevitable feature of electrical supply, although recent improvements in power-distribution systems and in motor protective-devices have reduced its frequency. When it occurs and a long column of water has to be stopped, there is bound to be some surge. This surge will take the form of periodic changes in pressure above and below normal. It has generally been found that if the valve which acts as a check against back-flow through the pump can be made to close while the water is still travelling through it in the normal direction, but at a decreasing velocity, the increases in pressure above normal are seldom very great, often not more than 5 or 10 per cent.

If the valve does not close until the water has reversed its direction of flow and has established a certain velocity in the other direction, the increases in pressure are very severe, often reaching several times normal pressure momentarily. This gives all the equipment and the piping in the station a severe jolt which is usually repeated several times with only slightly diminished force.

Selection of Check Valves. Water hammer is one of the inherent hazards of the electric drive. When electric motor-driven pumps are installed, a great deal of care and study must be given to installing a form of check

valve that will either be certain to close quickly, before the water has stopped running ahead through it, or else close very gradually, after the direction of flow has been reversed through it.

The ordinary check valve will do neither of these things. The special valves that will meet these conditions satisfactorily are exceedingly expensive in small sizes, say under 12 in. For small pipe sizes, the best economy is usually to use plain check valves, with adequate pressure-relief valves, to

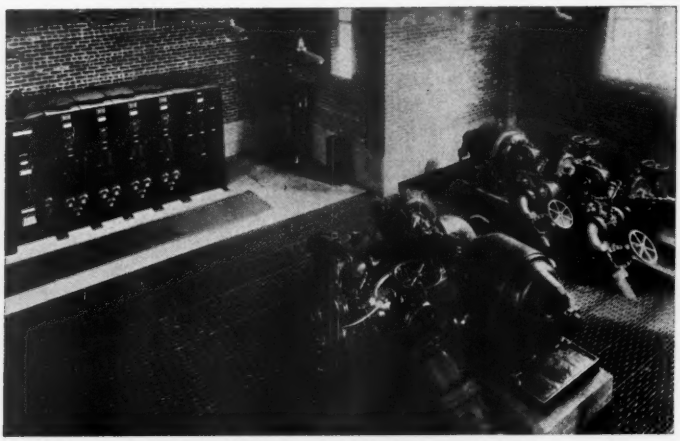

FIG. 4.— GROVE STREET WATER PUMPING STATION, MONTCLAIR, N. J.
Initial Pumping Units with Check Valves and Piping.

make the piping extra strong and anchor the piping and pumps solidly enough to stand the gaff. This, however, does not prevent the system-pressure disturbance.

For larger sizes it usually becomes more economical to use special check valves, carefully selected for the expected characteristics of the system. Sometimes these are operated by the system water-pressure and directly controlled by the water flow, sometimes they are operated by the system water-pressure but electrically controlled, sometimes electrically operated and controlled. Electric valves often require an independent power-supply, because the failure of the normal electric power is one of the circumstances that brings these disturbances into action.

In water-works stations, we have had our best success with hydraulically-operated valves. The recently installed valves in the Montclair station are controlled by the velocity head through them and give promise of being the best installation the writer has seen in this respect. They react quickly, and surely. They can be controlled easily by hand. Their operation can be adjusted simply when different action is desired.

DISCUSSION.

FRED O. STEVENS* (*by letter*). Mr. Greene's paper is of especial interest to me, since I have just completed two automatic-regulation jobs, one of the closed vacuum-switch type, and one employing float switches.

To help forestall the possibility of centrifugal pumps, automatically primed, running without water, it has been our practice to install a switch, either of the vacuum or float type as the case demands, so wired as to break the main pump-circuit, whenever the vacuum or the water level drops to a predetermined danger point. If, in addition to this precaution, a line from the vacuum pump is tapped into the eye of the impeller, it would seem that the possibility of a pump running without water would be very remote.

That a possibility would still remain, I am bound to admit, and I have often thought, without ever getting beyond the thinking stage, of installing some kind of a thermostatic control in the pump casing or the suction nozzle, designed to open the main circuit as soon as the temperature, due to running dry, rises a few degrees.

The really troublesome problem in connection with these automatic installations, at least to us who are dealing with relatively small units, is the control of the surge and the development of a valve for this purpose, at a price somewhere in proportion to the cost of the main units.

*Of Symonds & Stevens, Consulting Engineers, Boston, Mass.

REBUILDING NO TOWN RESERVOIR DAM.

BY W. GUY CLASSON.*

[*Read March 11, 1931.*]

The name of the dam discussed in this paper may seem strange. The story goes, that at the time of the original land grants, some 3 or 4 sq. miles were not included in any township. Later this section became Fitchburg and Leominster.

No Town Reservoir Dam and Dike were built in 1853 by an association of mill owners in Leominster to equalize the flow of Monoosnock Brook, which flows through this city. This association was known as the Leominster Reservoir Company, the members of which were assessed according to the height of their mill privilege, to maintain and operate this reservoir. The privileges in Leominster have 8- to 25-ft. heads, and the power plant of the Fitchburg Gas and Electric Light Company had a privilege in Fitchburg with 188-ft. fall.

In 1926, the city of Leominster acquired this reservoir as an additional water supply. The power plant in Fitchburg was purchased outright. This gave us Simonds Pond, and 60 per cent. interest in No Town Reservoir and Rocky Pond. The damages for water rights and interest in the reservoir controlled by the mill owners in Leominster, were settled on a progressive basis, *i. e.*, Leominster is allowed a certain amount of water over 5-year periods for 20 years, when the city will own the reservoir outright.

No Town Reservoir has an area of 237 acres, 54 of which are in Fitchburg. The capacity is 681 m.g., with about 5 sq. miles of drainage area. Several hundred acres of land on the watershed have been purchased. The original dam, about 600 ft. long, was first built with a wooden flume to control the draft of water, and a spillway 18 ft. wide was provided at the westerly end of the dam. The embankment was constructed of sandy earth available in the vicinity, and apparently no special precaution was taken to bond it into the natural earth, or make the embankment particularly impervious. No core wall was provided.

In 1876, the flume was removed and replaced by two 24-in. cast-iron pipes, a brick gatehouse was built, and cut-off walls placed around pipes. In 1889, the spillway was lengthened to 24 ft. In 1893, it was again lengthened to 37 ft., and the height of the main dam and dike was increased to provide $2\frac{1}{2}$ ft. more water. Later, the spillway was increased to the present width of 60 ft. Although the spillway had been increased several times, the channel below the spillway was never widened, and was not sufficient to take care of freshet flows.

In 1928, Charles W. Sherman of Metcalf and Eddy, was employed

*Superintendent of Water Works, Leominster, Mass.

to make a study and advise the best method of improving conditions. Mr. Sherman recommended that the channel be increased to full width at once, that the cross-sections of dam and dike be increased by grading the downstream sides on 2 to 1 slopes and the water face on 3 to 1 slopes paved with stone at least 12 in. thick.

In the fall of 1928 we started excavating the channel, using the ma-

FIG. 1.— FACE OF OLD DAM SHOWING PAVING.

terial, which was hardpan, to grade the downstream face of dam. All work was done by hand, employing water-works men. Figures 1 and 2 show the upstream face of the dam before and after reconstruction.

The cost of rebuilding the dam during 1928 to 1931, is shown in the following schedule:

Excavating channel and grading downstream slope of dam in 1928..	$3 455.42
Concrete cutoff walls, grading and paving small section in 1929....	6 351.57
Steel piling in place, grading and paving balance of dam in 1930.....	16 692.73
Total.....	$26 499.72

These costs include labor and material, engineering, rental of machinery and use of water-works equipment.

In 1929, a small section of the water face of the dam was rebuilt, as an experiment. A concrete cutoff wall was placed at the foot of the old dam, excavating about two feet into hardpan. The cutoff wall extended 3 ft. above ground level. From the top of this wall grouted paving was placed on a 3 to 1 slope and then capped with a 4 to 6-in. concrete slab. A concrete

wall 5 to 6 ft. high was then placed along the top line of the dam. This formed a substantial core wall. Grading was then done on a 3 to 1 slope and paving with field stone 12 to 18 in. thick.

In 1930, we changed our methods and drove steel piling ($\frac{1}{2}$-in. web weighing 36 lb. per ft.) 10 to 20 ft. long into hardpan, the top of the piling being left about 1 ft. above ground level at the foot of the old dam. On top

FIG. 2.— FINISHED DAM LOOKING WEST.

of this piling was placed a concrete wall about 3 ft. high. We then proceeded along lines similar to those of the previous year, except that galvanized wire mesh was placed between grouted paving and concrete slab.

The pile driving was done by Buck-McDonald of Boston. The contractor was given 28 working days to do the work but completed it in 11 working days. Seventy-eight tons of steel were used.

ITEMS IN THE OPERATION OF NEW YORK CITY'S WATER-SUPPLY SYSTEM OF INTEREST TO THE SUPERINTENDENT.

BY WILLIAM W. BRUSH.*

[*Read February 11, 1931.*]

Each water-supply system has one or more special operating problems as well as a number of operations that are common to many systems. The way in which the special problems are solved may have a general interest, but there is no question as to the interest of superintendents in what is done with the common problems. It is the writer's intention to set forth as briefly as he can New York City's practice as to certain items, not with the idea that it is the most desirable, but rather to promote thought and discussion.

WATERSHEDS.

Purchase of Land for Protection of the Water Supply from Pollution. Since chlorination has been available for disinfection of water it has not been considered necessary to purchase land beyond a marginal strip about 500 to 1 000 ft. wide around each reservoir. Unless one can control the entire watershed, there appears to be little advantage even in securing control over the land bordering the streams. There is a decided question as to the wisdom of rendering large areas uninhabited because they are part of a watershed, when these areas might be kept available for habitations while the wholesomeness of the water supply remains amply protected.

Patrol of Watersheds. It is only good housekeeping in water supply to keep the area from which the water is secured in as clean a condition as practicable. The desirable frequency of inspection depends upon the character of occupancy. The intelligence and diplomacy shown by the inspector are usually more important than the number of times he visits any location. At present, our trend is towards fewer inspectors while providing more automobiles for their use. The inspectors are also charged with preventing as far as possible unlawful entry on the water-supply property, bathing in the reservoirs and the committing of nuisances. The number of watershed inspectors and assignments are as follows:

Watershed.	Area Sq. Miles.	Number of Inspectors.	Number of Automobiles for Inspectors.
Schoharie	314	3	3
Esopus	257	6	none
Croton	375	15	7
Bronx and Byram, including Kensico Reservoir	22	5	1

*Chief Engineer of Water Supply, Department of Water Supply, Gas and Electricity, City of New York.

Bathing, Boating, Fishing and Ice Cutting. The New York State Department of Health does not prohibit bathing in waters from which public supplies are taken and looks to the community using the water to adequately purify it. Bathing is of course prohibited when the city owns the shore of the stream or reservoir. Bathing, however, takes place in several bodies of water tributary to the New York supply, and at some locations public bath-houses are established. In the writer's opinion it is probably not unreasonable for the state to allow persons residing on a body of water to bathe therein even if the water later flows into a water-supply reservoir, but the development of such bathing as a business should be prohibited.

Boating, fishing and ice cutting are, by law, allowed on the city reservoirs under regulations established by the department. A department permit is required to keep a boat or cut ice on a city reservoir, but no permit is necessary to enter water-supply lands for fishing from the shore or from a boat for which a permit has been issued. No renting of boats is allowed.

Sewage Collection and Disposal. When the development of a community reaches the point where a public sewer-system is desirable to protect the water supply under the law, the city in agreement with the local officials builds and operates the sewer system, extending the house laterals to the buildings. The owner must make the connection at his expense. Individual residents and institutions must provide for sewage disposal in a manner satisfactory to and under plans approved by the department. For the larger plants the State Department of Health must also give its approval.

Chlorination of Stream Flow. Where tributary streams are sufficiently polluted by bathing or by density or character of resident population, the department installs a chlorinating plant which will treat all but the flood flows and thus offset the greater part of the pollution. These small plants are visited by an employee once a day or at a longer time interval and, therefore, cost very little for operation. It should be kept in mind that all the water thus treated is later chlorinated at least once and usually twice before being delivered to the consumer.

Taxes. On water-supply lands and structures other than aqueducts located outside the city limits, the city pays taxes amounting to over $1 000 000 annually. Naturally, the views of the local tax assessors and those of the city's representatives do not agree as to the assessments, and it has therefore been necessary to generally protest and frequently litigate such assessments.

Reforesting. The state furnishes free pine seedlings for reforesting, and some 30 000 to 40 000 of these seedlings are annually planted on water-supply lands. To date about 6 000 acres have been covered, representing about one quarter of the city's holdings of land above flow lines.

Copper Sulphate Treatment. The miscroscopic growths are controlled by copper sulphate, which is generally applied when and as required by using launches and boats with outboard motors. In the late fall a special examination is made of the water in the reservoirs which feed directly into

the aqueducts to determine if the growths are such as to require treatment as a precautionary measure before the ice forms. In a reservoir like Kensico it is our usual practice to make a treatment in December of each year. In the Croton system where the water must be drawn only from one body of water, Croton Lake, a plant has just been installed so that copper sulphate solution can be pumped into the water on the upstream side of a small dam. An 8-in. wooden pipe with small copper tube outlets is being used. This plant has not been in use long enough to demonstrate its efficiency.

Weed Growth on Exposed Areas. As the various reservoirs are drawn down, a rapid growth of vegetation takes place on the exposed areas. Many years ago, an effort was made to burn or otherwise destroy this growth before the area was again flooded but without much success. No detrimental effect on the quality of the water has been noted due to the water later flooding the exposed area.

Aqueducts.

Cleaning of Interior. Virtually no interior cleaning is done on the aqueducts. The siphon forming that part of the Croton Aqueduct which is under the Harlem River has not been cleaned for thirty years. Chlorination of the water helps to first remove and later prevent growths forming on the sides and bottom of the aqueducts.

Patrol. It is general practice to patrol the various aqueducts about once a month, although certain sections are examined more frequently.

Stopping Leaks. Fortunately, there are few leaks on the aqueducts except on the old Croton Aqueduct. The latter is structurally weak, as the sidewalls are not sufficiently massive to withstand the overturning force of the water without the support of the earth backing. To stop a small leak the introduction of a mixture of cement and sawdust into the water flowing through the aqueduct will usually be effective. When it is necessary to close a crack by emptying the aqueduct, the use of lead wire about $\frac{1}{8}$ in. in diameter caulked into the crack has been found far more effective than lead wool.

Crossing by Other Structures. Where it is necessary to have a sewer or water pipe cross an aqueduct, special attention is given to the design to make certain that the pipe crossing the aqueduct is adequately supported so as to minimize likelihood of breakage. For sewer crossings, cast-iron pipe with lead joints encased in a concrete envelope or steel pipe is required for the full width of the right of way. A charge of ten dollars per annum for each pipe crossing is made.

Aërators. At Ashokan Reservoir and also at Kensico Reservoir outlet, there are aërators, each with about 1 600 nozzles. At Ashokan, all the water amounting to 600 m.g.d. is passed through the aërator while at Kensico about two thirds is aërated, this amount being the capacity of the aërator. The Kensico aërator operates throughout the year while the Ashokan

aërator is shut down between December 1 and the following April to prevent ice formation. The aërators are a beautiful part of the Catskill system and break up some of the more fragile microscopic organisms but are not essential for delivering a satisfactory water.

Chlorination. All water delivered to the consumer is chlorinated at least once, except in the case of certain well supplies where the conditions and past history of the supply make chlorination appear unnecessary. The usual rate of application is about 2 to 3 lb. per m.g., although at some points it is as low as ½ lb. and at times nearly as high as 6 lb. Generally, the rate of application is such as to maintain a residual of 0.1 p.p.m. of free chlorine. In a well water, a lower content of free chlorine is usually necessary to avoid chlorine odor or taste. There is no odor or taste of chlorine in the water supply except under unusual circumstances. A higher rate of chlorine application is made when microscopic organisms are sufficient in number to cause a taste in the water. Tastes as well as causative organisms are destroyed by chlorine.

Double chlorination of all surface supplies is a practice which the writer thinks is most desirable as a safeguard for the consumer. No matter how carefully a chlorination plant is operated there are likely to be short periods when the treatment either ceases or is below standard. Also some bacteria which escape destruction at the first plant will be destroyed at the second. With chlorine costing in ton containers about 1.7 cents per pound plus freight and handling charges at destination, the cost of double chlorination is almost negligible. The provision of spare chlorination equipment at every plant, so that machines will be available to replace any in need of repairs, is considered essential for safety.

Distribution.

Reservoirs. All distribution reservoirs are gradually being enclosed by wire-link fences of the so-called non-climbable type from 6 to 7 ft. high, but without barbed wire on the top. The omission of the barbed wire is due largely to the objectionable appearance of this wire. There have been two suicides in reservoirs so protected, the period covered being about four years. Previous to erecting the high fence around one of our reservoirs there were one or more suicides each year in that reservoir.

The sea gulls flock from the polluted waters of the bay and rivers to the reservoirs from August until the following April. Permit to kill the gulls has been obtained from the U. S. Government for two of the reservoirs and some success secured in driving the gulls away.

On the mains that connect gatehouses with the distribution system and where there are several parallel mains, the valves are equipped with electric motors, and float indicators show the main upon which a break has occurred.

Consideration should be given to the possibility of making either daily

or emergency use of a higher-level reservoir for a lower-level service. The practice of using an intermediate service-reservoir in Brooklyn to reduce the draft on the Catskill Aqueduct Tunnel for the low service has been adopted for the past two years with very satisfactory results. This reservoir, which is known as Mt. Prospect Reservoir, has a flow line at el. 202, a depth of 20 ft. and a capacity of 20 m.g. It is filled by water from the Catskill system, and this system also supplies a large portion of the low service with a hydraulic gradient of about 150 ft. By delivering water from Mt. Prospect Reservoir into the low service during the morning and afternoon periods of high draft with a rate of from 30 to 40 m.g.d., the Catskill Tunnel gradient in Brooklyn is maintained about 10 ft. higher than would otherwise be the case, and expensive pumping for the high service supplied by the Catskill system has been avoided. About two thirds of the reservoir contents are discharged during the daylight hours, and the reservoir refills during the night hours.

New Mains. When an extension to the distribution system is requested, and there is one house for each 150 ft. of main, an additional allowance of 100 ft. of main being made for each extra family in a house, the extension of the main is usually authorized and the cost is met by the sale of corporate stock, the property owner making no direct payment for the installation of the main.

To accelerate the extension of mains and thus meet the needs of builders, the department purchases and stores in its yards the necessary 6-, 8- and 12-in. pipe with the appropriate fittings, valves and hydrants and contracts under public letting for the laying of such pipe. All purchases are on the basis of department standards, the same hydrant standard having been used for about twenty-five years and the same valve standard for a somewhat shorter period. Corporation stops or taps are purchased by contract and inserted in the mains by department tappers at a fixed price to the property owner for each size of tap, starting with the $\frac{5}{8}$-in. size. The minimum service size is $\frac{3}{4}$ in., and the services are put in for and at the expense of the owner by a licensed plumber.

For the past year the cast-iron pipes have been lined with cement coated over with coal tar or asphalt applied hot. It is expected that this year a cold asphalt coating will be applied. This covering of the surface of the cement by a substance insoluble in water prevents hard water developing in long mains with slow flows and probably greatly increases the effective life of the cement lining in preventing corrosion.

Under the specifications for pipe that are to be used this year, the department permits "Universal" and "McWane" pipes to compete, having previously included centrifugally-cast pipes. It requires, however, that all the pipe have the same barrel thickness as Class B-AWWA. Bids will be compared on the resultant price per laying foot, the quotation to be the price per ton. Special castings or fittings are to be only bell-and-spigot type.

In laying water mains and also in effecting repairs, special attention is

paid to disinfection of the mains with chlorine. Chloride of lime is used for the smaller mains, a suitable amount being placed either in each length or else on the downstream side of the valve through which the water passes in filling the main. For larger mains, liquid chlorine is frequently used. As far as possible the laboratory reports on the sanitary condition of the water in new mains before they are placed in service. Where mains are placed in

CAST IRON HEMISPHERICAL DRAIN BASE FOR HYDRANT.

Capacity of base 2½ times water content of hydrant standpipe.

built-up sections, each length is washed out and swabbed with a chlorine solution before being laid.

In laying mains, the department has for many years used only 6-, 8-, 12-, 20-, 36- and 48-in. valves. On intermediate-size pipes the next smaller sized valve is used and on 48-in. mains, 36-in. valves are placed.

Hydrants. In setting hydrants, it has been our practice in rock trenches and clay soils to connect the hydrant drain to the sewer, using a lead-lined iron pipe. When the soil will absorb the water, a blind stone drain was formerly placed either under the hydrant or alongside it. Difficulty was experienced, however, in making certain that contractors place the broken stone for the drains; department men also sometimes slighted this work. Therefore, a hemispherical cast-iron dome, holding 2½ times the barrel

contents of the hydrant is now being used as a combined support and drain for the hydrant when it is not connected to the sewer. (See accompanying figure.)

Maintenance and Operation. The maintenance and operation of the distribution system is based on employing repair companies of which there are several in each borough. Each repair company has its own headquarters and the majority maintain an emergency force during the two night watches. Gate-operating trucks are assigned to each borough. In Manhattan and the Bronx, where rock-filled streets make the location of leaks very difficult, there is a separate organization to locate hidden leaks on the water mains. A special force is also assigned to the operation and lubrication of the large valves.

The fire hydrants are inspected by the firemen in the fall of the year, and the hydrants reported out of order are repaired by the water-department force. In Manhattan and the Bronx, special gangs, composed of a machinist's helper, laborer and light truck, are constantly examining and repairing hydrants which are used very extensively by the street-cleaning force.

Pumping. The only item of general interest in our pumping practice is the definite trend towards the substitution of electric power for steam on the basis that the cost is about the same and that electric drive is considered more satisfactory in built-up sections than steam power.

High Pressure Fire Service. This service is operated by the water department, the pumps being started on the alarm being received and the pressure held at 125 lb. per sq. in. in Brooklyn and at 150 lb. per sq. in. in Manhattan until a higher pressure is requested by telephone message from the fire department. The running of the stations is based on orders received from the fire department as long as the service is needed.

Conclusion.

There are numerous other items which might have been included but which would make the paper too long. The writer hopes that the recital of these items in the New York City practice may be helpful to some of his fellow water-works operators.

WROUGHT IRON.

BY JAMES ASTON.*

[*Read September 25, 1930.*]

In September, 1917, a Committee on Service Pipe of this Association reported as follows:

The experience of water-works superintendents as indicated by the returns and as shown by previous discussions in meetings of the Association is decidedly unfavorable to steel pipe, and while undoubtedly, many of those who argue in favor of wrought-iron pipe may be unfamiliar with anything else and believe in it because they were taught to believe in it, there is certainly a large number of reliable men who have had extended experience with both materials who are firmly convinced that wrought iron is much superior to steel, and it would seem that there must be some good reason for that belief.

The intervening thirteen years in the author's opinion, have not altered the factors that prompted the above report. They have, however, seen some important transitions in the manufacture of wrought iron. It is opportune, therefore, that the author review the general service conditions and economic situation as related to wrought-iron pipe for water conveyance, and in particular, that he tell something about developments in manufacture which are having a profound effect upon the industry and which will no doubt bring about a readjustment of past ideas in the selection of pipe materials.

The problem of the water-works engineer is to select pipe from among the several classes of material available. Considering the numerous factors involved, and their extreme variability, the problem is of major importance. The material must have the physical properties,— strength and ductility,— to withstand the pressures and load stresses of service and of installation. It must lend itself to practicable methods of fabrication and erection. It must at times withstand the added stresses of expansion and contraction or the fatigue of shock and vibration, influences that are often lost sight of and are particularly pronounced on shipboard and in railroad service. Above all, it must resist in suitable degree the corrosive, and sometimes erosive, action of the fluids being conveyed, or of natural or created soil and atmospheric influences. If the water-works engineer were dealing solely with one soil or water condition or one type of service, or could choose without regard to the initial cost of material, his problem might be relatively simple. But the basis of selection is the dollar; not the initial cost of the pipe, but the cost of service as represented by capital expenditure, charges attached thereto, and subsequent items involved in maintenance or replacements.

*Consulting Metallurgist, A.M. Byers Company, Pittsburgh, Pa.

The basis of selection may be different for each community or source of water supply; furthermore, it may vary in relatively small local areas, due to variations in soil or service conditions.

As a rule, the engineer has a fairly clean-cut problem so far as fabrication and service stresses are concerned. They are known or calculable and can be met by determined physical characteristics of the several available materials. Not so with the all important and vital question of corrosion. Corrosion involves a multiplicity of factors; some inherent in the pipe material, others linked to soil or water characteristics. Unfortunately, not only because of the number and variability of these factors, but also because of marked upset of expected conditions by seemingly minor influences that disturb the normal balance, corrosion testing does not parallel the determinative exactness of usual physical testing. Acceleration of tests, in order to secure quick-time results as a basis of selection, has thus far been of little value in diagnosing the relative merits of materials and has fostered propaganda that has clouded, rather than cleared, the situation. Experience still remains the basis for selection of materials for general corrosion service.

Water supplies are exceedingly variable in their corrosive effects, even within the composition range of potable quality. There is acceleration of the normal effect with aëration of the supply, or in hot-water service. We have come to recognize that film effects, caused by deposition of certain ingredients in the water, are important factors in retarding corrosion. Thus hard waters are often better from a corrosion viewpoint than soft ones of a higher degree of chemical purity. The problem would be simplified if water supplies could be universally and effectively treated, de-aërated, or given a film-producing power. Unfortunately, such remedial measures appear to be of only limited applicability and effectiveness, due to impracticability or expense of installation, difficulty of supervision and maintenance and unreliability of results. Frequently, the water-works engineer must accept conditions as he finds them and meet the situation with that material which in his judgment and experience will render service at the minimum cost during the life of the structure.

With such a multiplicity and variability of factors, it is little wonder that we meet differences of opinion. There is no set formula for human judgment; for similar installations it may lean towards high initial cost with long life and low upkeep, or to low cost with relatively short life. Naturally, also, there is a partisan stamp to the arguments of manufacturers of the several available materials, and an atmosphere of controversy may be expected in that widely overlapping field where several of the materials may function with more or less satisfaction. It is not the purpose of the writer to feed the flame of controversy, but to attempt to clear up the apparent ambiguity regarding wrought iron, and to bring forward certain developments in the problem of manufacture.

The pipe materials available in the field of water distribution, in the general order of cost, are copper, lead, brass, cast iron, wrought iron and

steel. Wrought iron is one of three types of ferrous products available, and in its general properties and applicability it is more closely aligned with steel than with cast iron. In short, by characteristic refining processes, wrought iron and steel have been brought to a relatively pure ferrous condition as contrasted with the initial pig or cast iron from which they are produced. As a result, they have higher tensile strength and ductility than cast iron. Wrought-iron and steel pipe are, in general, made by a rolling and welding operation. Both will stand relatively high service pressures and shocks, are adapted to screw-joint methods of installation, and may be bent hot or cold. Wrought-iron and steel pipe are made to the same manufacturing standards and are subjected to virtually the same test requirements.

Apart from these similarities, however, the fundamental characteristics and differences of wrought iron and steel are well recognized by metallurgists. While the same basic materials and reactions enter into their production, wrought iron exhibits on fracture a distinctly fibrous structure, while steel is decidedly crystalline. Chemically, wrought iron is found to have a highly refined iron base, while steel is usually less free from carbon and associated metalloids. Under the microscope, this refined base metal in wrought iron is shown to be intermingled with particles or threads of slag, while steel is free from this characteristic. Differences in properties of the two classes of materials are dominantly related to this feature.

Outstanding characteristics of wrought iron, which are the basis of its employment, are welding and threading quality, resistance to shock and vibration and resistance to usual corrosion influences. Opinions differ regarding any special merit which wrought iron may possess along the lines just enumerated. Nevertheless wrought iron has as its supporters a substantial body of experienced users. It is for service requirements where the service characteristics previously mentioned are dominant that wrought iron finds outlet in railroad, oil-well, public-utility, marine, industrial and domestic fields, in spite of a higher price than corresponding steel commodities. Opinion of users of pipe is adequately expressed in the quotation from the committee report of 1917 which forms the opening paragraph of this paper.

Wrought iron is the oldest ferrous metal known to man. Authentic samples from Egypt and India date from several thousand years before Christ. Manufacture was on a fairly well established plane in Egypt, India and medieval Europe a thousand or more years before the Christian era. The noted pillar of Delhi is generally assumed to date from 400 A. D., while the use of iron was general in Europe at the time of the Roman Empire. Early manufacture progressed through centuries of primitive production, and the limitations of these methods resulted in a material conforming to the general characteristics of what we now classify as wrought iron. Almost to the end of the eighteenth century, production was, in sharp contrast to present-day procedure, by direct reduction of the ore to a malle-

able product in a single operation. Output was necessarily limited by the general crudity of equipment, by the inadequacy of materials available, by labor conditions involved, and by the forced selection of only the purer grades of ore.

Another epoch in the iron industry dates from Cort's invention in England in 1784 of the puddling process. In conjunction with the previously developed blast furnace, it made possible greater flexibility of operation, latitude in selection of raw materials, control of product, expansion of output and centralization of manufacture, meeting thereby the growing needs of advancing civilization. The product was wrought iron, conforming in all essentials to that of the more primitive eras. It made for itself a reputation for durability which is exemplified by many records in European and American Colonial history.

The "Age of Steel" may be said to date from Bessemer's great invention of 1855. In this age we have seen construction methods revolutionized, the mechanization of industry, and tremendous advances in civilization; unfeasible, no doubt, except by methods which have made possible an expansion in the use of iron in the United States from 55 lb. per capita in 1850, to approximately 1 000 lb. today. With the departure in method, came a departure in material characteristics; a significance not noted in the earlier days to the degree that we now recognize.

The metallurgist today fully realizes that the fundamental differences between wrought iron and steel have been brought about primarily by the end temperatures attained in the processes of manufacture. In steel making, the slag resulting from the refining operation separates out, because the final temperature of the heat is above the fusion point of the metal and of the slag. In the puddling operation, on the contrary, the final temperature is lower than that required to fuse the metal. It solidifies, therefore, to a pasty mass in the bath of slag, and is disintegrated therein by the manual effort of the puddler. In confirmation of this distinction, the following definition of the American Society for Testing Materials* may be quoted: "Wrought iron is a ferrous material, aggregated from a solidifying mass of pasty particles of highly refined metallic iron, with which, without subsequent fusion, is incorporated a minutely and uniformly distributed quantity of slag."

Until about sixty years ago, marking the application of the open-hearth furnace to steel making, the puddling process produced wrought iron primarily because of the temperature limitations resulting from inability of existing furnaces to reach the heat needed for final fusion. Today, the puddling furnace and practice survive without fundamental changes due to recognition of the distinctive and useful characteristics of wrought iron, and because there has been a general lack of success in the contemporary period in reproducing the desired characteristics by a more efficient procedure.

*Triennial Standards, 1930, p. 464.

I shall not burden you with a lengthy description of the puddling process. It has of necessity been associated with labor of a skilled character, and with small outputs. The general heat is about 600 lb. and requires the hard work of two puddlers for $1\frac{3}{4}$ hours to produce the characteristic spongy balls of refined iron, each about 200 to 300 lb. in weight. Five heats, totaling 3 000 lb., is the turn output for two men.

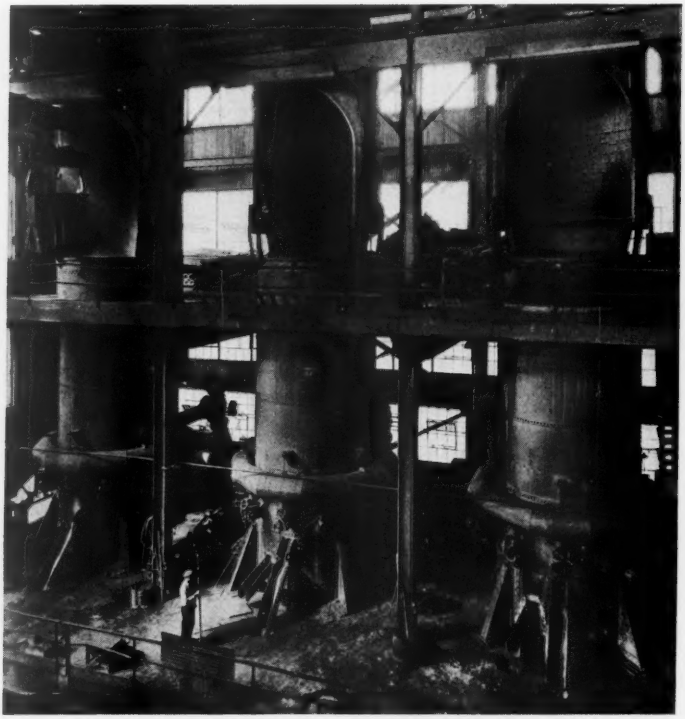

FIG. 1.— THREE 18-TON CUPOLAS.
These furnish a continuous supply of molten metal.

Little wonder that wrought iron thus produced should be a specialty and a relatively costly commodity, when we compare hand-puddling with steel-making practice. Much capital and effort have been expended in developing a "mechanical puddler," with generally unsatisfactory results; at least to the extent of causing abandonment of hand puddling as the dominant process in the manufacture of high-grade wrought iron. Yet the industry has been fully awake to the necessity of developing improved methods, if it is to be expanded to a material degree. High cost and limited tonnage have lost or curtailed certain markets, and it is realized that these

factors represent the crux of the economic problem of the wrought-iron industry. Since 1915, the A. M. Byers Company of Pittsburgh, Pa., a leading manufacturer of wrought-iron pipe, has been developing a process for wrought-iron manufacture which about three years ago reached the commercial stage of production at Warren, O. This operation was described in a paper* before the American Institute of Mining and Metal-

FIG. 2.— TWO 10-TON CONVERTERS.
These remove the impurities from the molten metal.

lurgical Engineers in September, 1929. The present paper describes an operation in the Pittsburgh district, which began less than two months ago, and has superseded the one at Warren. The latest operation is in an entirely new plant erected for the purpose, with an initial capacity of about 18 000 tons monthly; in contrast with the Warren output of 4 000 to 5 000 tons monthly.

After a prelude of many years of study of the characteristics of quality wrought iron and the essential steps in its manufacture, the key to the problem was found to be in reducing to minute particles the refined base metal, and its incorporation with the proper amount and quality of slag. The process, as developed, conforms to puddling in all its essential steps. These, however, are separated and carried out in pieces of equipment best suited to the several phases, as contrasted with a combination of all in one

*"A New Development in Wrought Iron Manufacture," by James Aston. Am. Inst. Min. & Met. Engrs. Cleveland Meeting. Fall Meeting. Iron & Steel Division, *Publication No. 228.*

furnace in usual puddling practice. The result is control of operations, magnitude of production, uniformity of product, and minimizing of labor to a degree not attainable in puddling.

Pig iron is melted continuously in cupolas. This molten metal is blown in a Bessemer converter to refined iron conforming to the base-metal characteristic of wrought iron. The blowing time is approximately ten minutes

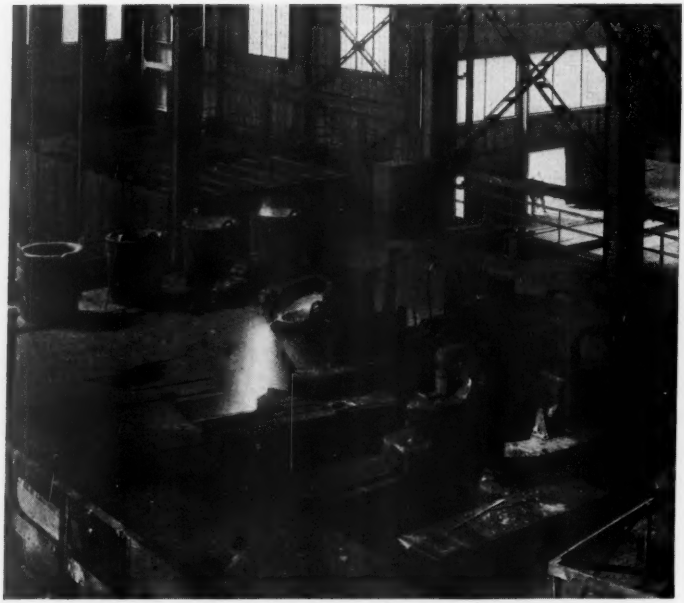

FIG. 3.— PROCESSING.
The key operation in the manufacture of wrought iron.

for the 10-ton heat, and the converter will produce the refined metal in a steady supply on a 15-minute cycle. (Figures 1 and 2).

Next follows the key operation of the process, that of disintegration and slag incorporation. It is carried out in a set of unlined cups or thimbles, resting upon cars that may be moved to the several stations representing stages of the process. The cups are cubical, about 6 ft. square and 6 ft. deep. One or more is supplied with about 4-ft. depth of slag of wrought-iron characteristics, melted in a tilting open-hearth furnace. Into each cup of slag, three tons of the refined metal are poured in a steady stream, the approximate time being three minutes. Considerable boiling and gas evolution will be noted in the cup, but there is no violence in the reactions. Immediately upon completion of the pour, and quieting of the reactions, the cup is picked up by a crane, and the large surplus of slag is decanted

into an adjoining empty thimble, and is available for the following heat of metal, after additions of such make-up slag as may be needed from time to time. (Figure 3).

The decanted cup contains a solidified, spongy mass of metal, intermingled with liquid slag, conforming in all respects and surpassing in many, the product of the best hand-puddling practice. This "ball" is transferred

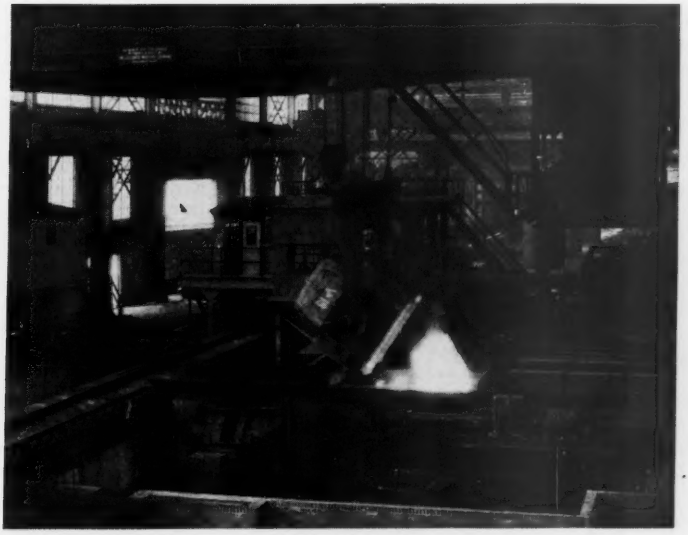

FIG. 4.— THE BLOOM PRESS.
In this 900-ton electrically driven press is compacted the 6 000-lb. bloom.

to a powerful press; and being at a welding heat, is compacted to a solid, rectangular bloom about 18 x 22 x 72 in.; accompanied by proper ejection of surplus slag during the squeezing operation. The bloom is immediately rolled to appropriate billet sizes on a large electrically driven, reversing, blooming mill. (Figure 4).

In order not to burden you with technical detail, it will suffice to state that the mechanism of this apparently simple process of breaking the iron into minute particles is founded upon sound physico-chemical principles. The continuous solidification of the metal which is being poured into a bath of slag below its freezing point, results in liberation of enclosed gases with sufficient force to shatter the metal into a spongy mass which settles to the bottom of the vessel and is thoroughly intermingled with the liquid slag at a welding temperature. A uniform product, conforming in all particulars to the characteristics desired in quality wrought iron, is the result.

It is interesting to contrast the new operation with the previous one at

Warren and with hand puddling. The present ball weighs three tons; at Warren it was one ton, while in hand puddling the weight is 200 to 300 lb. Naturally, all accompanying rolling-operations conform to best mill practice, rather than the costly small-scale methods associated with hand puddling.

That the highest standards of quality wrought iron have been met has been confirmed by numerous users in government, railroad, marine and industrial lines after careful investigation. In every instance the wrought iron produced by the new process has been accepted under standard wrought-iron specifications, such as those of the Federal Specification Board (of the United States of America) and the American Society for Testing Materials. The following quotations are taken from a report of the United States Bureau of Standards,* following an extensive investigation:

The structural features of the new process wrought iron are those which are generally conceded to be characteristic of wrought iron in general. The absence of "carbon streaks" in the new process iron is noteworthy. It is not to be concluded, however, that this feature is necessarily always present in hand puddled iron. The hand puddling process is not nearly so favorable for the complete elimination of carbon, however, as is the new process. The occurrence of occasional relatively large slag threads in the hand puddled iron was more frequent than in the comparison material according to the examination made.

Although not a primary object of this investigation, general observations made of the hand puddling process and of the new process seemed to indicate a close parallelism in the important reactions entering into the manufacture of wrought iron in the two cases. The similarity of the products of the two processes confirms this indication.

The wrought-iron industry has lost certain markets due to its handicap of output and cost. Many of you may no doubt recall favorable experiences with wrought iron in plate structures, such as tanks and large pipe; quite in line with the excellent service rendered in the Rochester water conduit. The new plant just described is rolling plate in a large modern mill; in addition it is equipped with mills for rolling rods, bars, light structural sections, sheet bars, and numerous other commodities other than pipe skelp, which has been the regular product of the company.

The needs of water distribution are particularly worthy of careful consideration. Deterioration of structures through the ravages of corrosion is a severe charge against the investment. It behooves the engineer to properly weigh his problem, and to fit wrought iron into its proper niche. It has had an important place in the past, in spite of manifest limitations. It will in the future occupy an increasingly important position, by reason of diversified products, tonnage, and lowered costs made possible by the new developments in the wrought-iron industry.

*Bureau of Standards Research Paper No. 124 — December, 1929.

PROCEEDINGS.

March Meeting.

Hotel Brunswick, Boston, Mass.

Wednesday, March 11, 1931.

President George H. Finneran in the Chair.

The Secretary announced the election by the Executive Committee of the following to membership in the Association:

Members: Cornelius R. D. Murphy, Onset, Mass.; Nelson Boardman, Danvers, Mass.

Associates: S. R. Dresser Manufacturing Company (Richard L. Rideout), Arlington, Mass.; Eureka Pipe Company, Lynn, Mass. (Frank Arnold Purdy, Malden, Mass.)

The President. The Constitution of the Association provides that at the March meeting a committee be elected to make nominations of officers for the coming year.

(On motion of Theodore L. Bristol, duly seconded, it was VOTED that the President be empowered to appoint a nominating committee.)

The President. I regret to announce that since our last meeting one of our members, Henry A. Symonds, has passed away. He was a member of our Association for twenty-five years, serving as editor for six years. Mr. Symonds was a valued member, faithful in attendance, earnest in his interest in all things pertaining to the Association. We regret his loss and shall miss him.

The President read a list of bills before the 1930 session of the Massachusetts Legislature that bear upon water supply.

A paper, "Practical Points on Operation of Pumping Stations for Town Water Works", was read by Charles E. Greene, Electrical-Mechanical Engineer with Metcalf & Eddy, Boston, Mass.

A paper, "Rebuilding No-Town Dam of Leominster Water Works", was read by W. Guy Classon, Superintendent of Water Works, Leominster, Mass.

(*Adjourned.*)

Special April Meeting.

Hotel Taft, New Haven, Conn.

Tuesday, April 28, 1931.

President George H. Finneran in the Chair.

The President read the following paragraph from a letter by Raymond J. Kenney, Chief Warden of the Department of Fisheries and

Game of Massachusetts, relative to the killing of gulls and terns polluting public water supplies:

You will undoubtedly be interested to learn of the operation of Chapter 21 of the Acts of 1931, which becomes effective May 13, 1931, and which allows the killing of gulls and terns which may be the cause of your pollution. While no permit is required for the killing of these birds as far as the State law is concerned, they are also under the protection of the Federal law. Due to the fact that they are migratory birds and under the Federal regulations, the authorities in charge of the water supplies can not kill these birds without first securing a permit from the Bureau of Biological Survey at Washington. To facilitate matters in this respect, I have completed negotiations with the Bureau whereby a number of permits have been issued to this division with authority to issue them to the various water departments in the name of the Federal Government, to protect water supplies as are menaced by gulls and terns. Therefore, if the authorities in charge of any reservoir where this danger of pollution exists will file an application in this office, directing it to Mr. O. C. Barnes, Supervisor of Permits and Claims, the matter will receive prompt attention and will eliminate the necessity of communicating with the Federal Government in connection with this matter.

Edward E. Minor gave a description of the new work of The New Haven Water Company, and Professor R. C. Hawley explained the extensive forestry operations of the Company. Both talks were illustrated with stereopticon views.

(*Adjourned.*)

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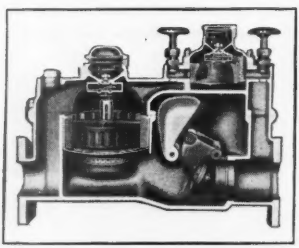

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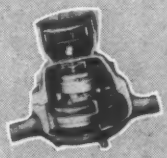

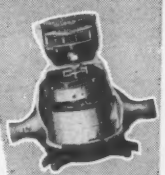

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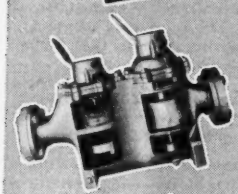
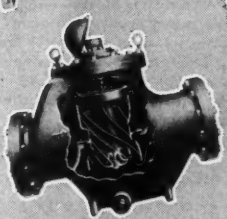
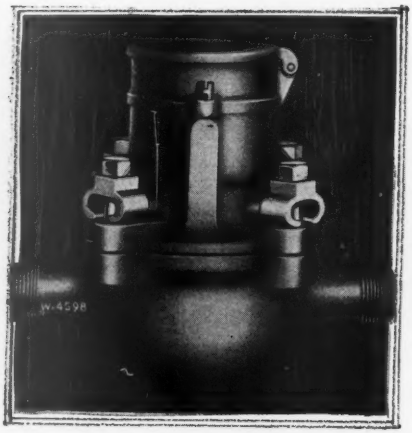




PITTSBURGH EQUITABLE METER COMPANY

Main Offices and Factories:
Pittsburgh, Pa.

New York Seattle Los Angeles
Chicago Tulsa Columbia
Dallas Kansas City Houston
Salt Lake City Davenport



A Modern Meter

. . . with advantages *worth having*

THE Worthington Disc Meter . . . the result of seventy-three years experience in meter building . . . offers definite advantages that promote satisfaction among water users and reduce the cost of water service.

1. **Improved Train Construction.** Monel Metal gears and spindles, accurately cut, combine sensitive action with accuracy and long life. Bearings are all rubber-bushed. Gears are integral with the spindles. The train may be quickly disassembled . . . simply by removing three screws. Either open train or oil-enclosed train is available.

2. **Reinforced Disc.** Sturdy construction, without a thrust roller, gives protection against breakage from water hammer . . . as well as from warping when hot water backs up.

3. **Frost Protection.** The Frost Clamp, developed by Worthington and available on Worthington Meters *only*, provides a real solution to the frozen meter problem, without resorting to cast-iron parts. As the water in the meter freezes, the clamps yield gradually. There is no shock or damage to the operating parts. Bent clamps can be replaced quickly at small expense. Water hammer will not break these clamps . . . a protection against flooded cellars.

Write for descriptive bulletin W-900-B3, or ask the nearest Worthington office for further information.

WORTHINGTON PUMP AND MACHINERY CORPORATION

Works: Harrison, N. J. Cincinnati, Ohio Buffalo, N.Y. Holyoke, Mass.

Executive Offices: 2 Park Avenue, New York, N.Y

GENERAL OFFICES: HARRISON, N. J.

District Sales Offices and Representatives:

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BUFFALO	CLEVELAND	DETROIT	KANSAS CITY	NEW YORK	ST. LOUIS	SAN FRANCISCO	WASHINGTON

Branch Offices or Representatives in Principal Cities of all Foreign Countries

PUMPS
All Sizes...All Types
For All Services
Any Capacity...Any Pressure
COMPRESSORS
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and Auxiliaries
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AUTOMATIC HEAT TREATING MACHINES FOR DRILL STEEL
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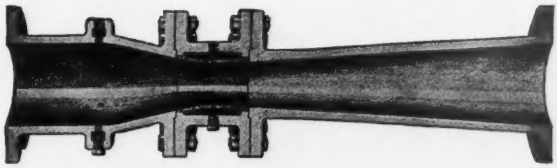
Literature on request

M-53

WORTHINGTON



The Venturi Tube

In 1891 the Venturi Tube was new. Those Engineers who knew of it hoped it would help solve the difficult problem of measuring the flow of water in large pipe lines.

In 1931 the Venturi Tube is accepted by Engineers as the most dependable and accurate means of measuring flow through pipe lines.

It may be worthwhile, however, to summarize a few of the important advantages of the Venturi Tube:

1. The flow is guided at all points.
2. The inlet and throat pressures are taken off at definitely fixed points; the annular pressure chambers assure the use of true average pressures.
3. The Venturi throat is accurately centered in relation to the rest of the pipe line.
4. The friction loss through a Venturi Tube is the lowest of any differential producer. The Venturi Meter, then, operates on a greater differential with less friction loss, and is a more powerful instrument with a wider measuring range than other meters.
5. The coefficients of the Venturi Tube have been definitely established in numerous calibration tests of tubes ranging in size from 1 inch to 8 feet in diameter.

VENTURI
"VENTURI" Registered

BUILDERS IRON FOUNDRY

"Builders of the Venturi since 1891"

9 Codding Street Providence, R. I.

FEW PARTS
EASY to REPAIR
.. that's what I like about Niagara Meters

Otto Parsons, Supt.
Painsville Waterworks
Painsville, Ohio

PAINSVILLE, Ohio Waterworks has been using Niagara Meters since 1893—thirty-eight years! Mr. Otto Parsons, Superintendent, recently told a Buffalo Meter Company representative that the reason he likes Niagara Meters is because of their few parts, ease of repairs and low maintenance cost.

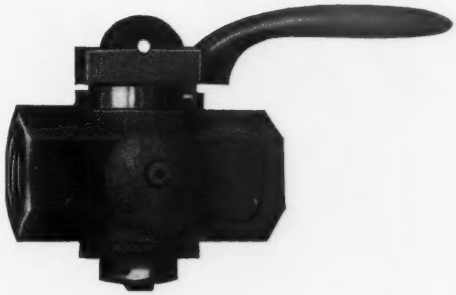
Because of the simplicity of construction and the correct application of engineering principles, Niagara and American Water Meters are everywhere meeting the exacting tests of time. When it is necessary to make repairs the cost of repair parts is extremely nominal.

Before you install water meters in your municipality, look at the long and favorable records behind Niagara and American Water Meters. This is your assurance that your city will get the most for its money.

NIAGARA (IRON CASE) and AMERICAN (BRONZE CASE) Water Meters

BUFFALO METER CO.
2889 Main St. Buffalo, N. Y.

For Maximum Service



Union Corporation, Curb and Waste Stops and Fittings. Always made of the highest grade bronze composition.

Constructed on sound scientific principles. Each plug solid except for waterway.

Ground and lapped to perfect bearing. Tested under 250 pounds water pressure.

Union Stops and Fittings are heavier and stronger.

Union Water Meter Co., Worcester, Mass.

Incorporated 1868

New York: 50 Church St. **Philadelphia: 424 Bulletin Bldg.**

UNION

Water Meters **Chronometer Valves** **Easy Turning Stops**
Pressure Regulators **Presses for Lining Pipe** **Alarm Signals**

SIMPLEX
VENTURI TYPE
METERS

Indicate the flow on a uniformly graduated dial.

Record the flow on a rectangular chart with uniform graduations.

Totalize the flow on a five-dial totalizer, so designed that the last figure can be read with accuracy on a much larger scale.

They may be furnished for floor stand, wall bracket or panel mounting.

May be used for pump discharge — gravity supply — in the Filter Plant — or Sewage Treatment Works.

Simplex Meter Registers may be checked at any time while in service by use of apparatus furnished as standard equipment — an exclusive feature of the Simplex Meter.

Simplex Meters are the choice of many who value quality and accuracy above initial cost.

Write for data.

Simplex Valve and Meter Co.

6757 Upland St., Philadelphia, Pa.

"WATCH DOG" WATER METERS

ACCURATE *DURABLE*

DISC, CURRENT AND COMPOUND TYPES

Inquiries Solicited

GAMON METER COMPANY

NEWARK NEW JERSEY

METERS

If you want a disc meter you will play safe with

THE
NASH

Our long experience in the business has made us thoroughly familiar with most of the conditions under which meters are required to work. The NASH is designed in the light of that knowledge, and is a meter to depend on.

National Meter Co.

299 Broadway, New York

Chicago Cincinnati Boston Atlanta
Houston San Francisco Los Angeles

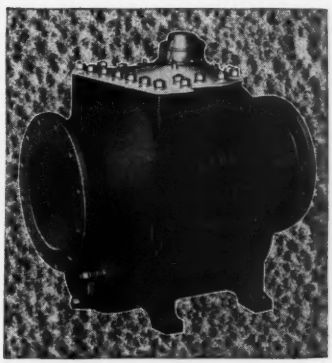
DURABILITY

—where millions cross with confidence!

Spanning the Hudson—this gigantic structure expedites transportation—millions of pedestrians and vehicles alike cross it regularly—cross with confidence. And why? It has stood the test of time—it's durable.

Likewise, now after a quarter of a century, more than a million Badger Water Meters in service continue to perform dependably.

The Badger Turbine is designed for heavy service such as railway standpipes, water supply mains, pump station meters, hydraulic elevators and the like. It will handle large volumes of water at a minimum loss of line pressure. It has the largest capacity in each size of this type due to large water ways and minimum of friction in all working parts.



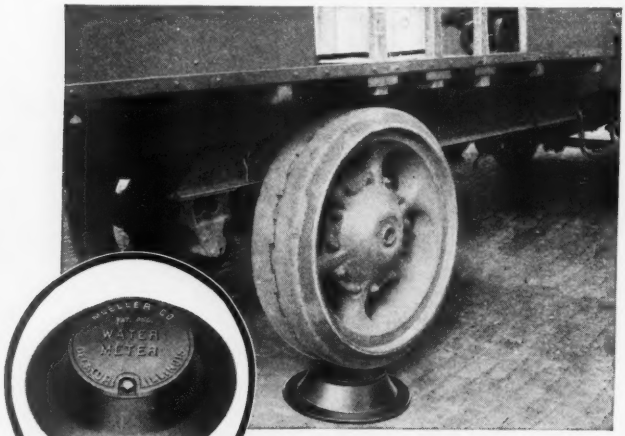
Ask for Bulletin No. 700.

BADGER METER MANUFACTURING CO., Milwaukee, Wisconsin

BRANCH OFFICES IN

NEW YORK CITY SEATTLE, WASH. SAVANNAH, GA. CHICAGO, ILL.
CINCINNATI, O. LOS ANGELES, CAL. WACO, TEXAS
KANSAS CITY, MO. PHILADELPHIA, PA.

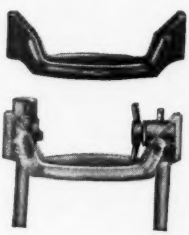
BADGER Water METERS



The story in a picture

The above shows how well a MUELLER Meter Box Cover stands up under a five ton truck. Not so bad, what?

You see, factors of safety—convenience—and economy now point to the curb as the most practical place for water meters. The MUELLER Meter Box and Yoke was designed to meet the changed conditions of this new location. Well designed patterns have made possible a casting in the best gray iron of more than usual strength. The special worm type lock simplifies the problem of cover opening. When it is necessary to remove the meter, uninterrupted service is obtained by simply dropping a nipple in place and tightening it.



This is but one of the many refinements that MUELLER is able to offer through their seventy and more years of familiarity with waterworks problems.

MUELLER CO. (Established 1857) *Factory:* Decatur, Illinois. *Branches:* New York, Dallas, Atlanta, Los Angeles, San Francisco, Chicago. *Canadian Factory:* MUELLER, Limited, Sarnia.

Write today for complete information on the MUELLER Water Meter Box Cover and Yoke and other Waterworks Specialties.

MUELLER

Trade-Mark Reg. U.S. Pat. Office

ATTENTION
WATER WORKS
OF NEW ENGLAND

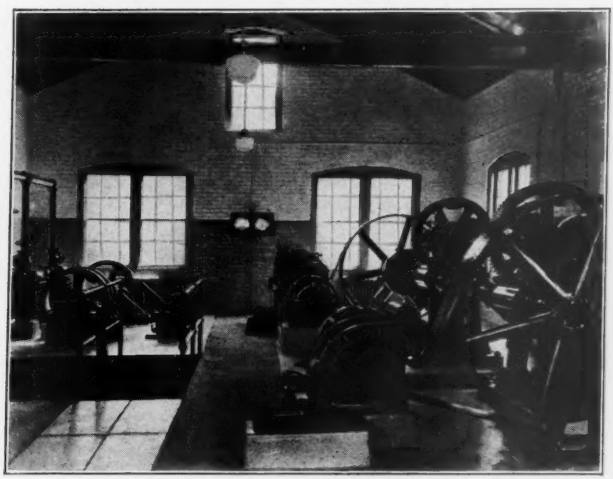
A few reasons why we solicit your inquiries on Water-Works Pumping Equipment:

First. — We make a specialty of furnishing Complete Water-Works Pumping Units of all types.

Second. — Total responsibility of installation is assumed by us.

Third. — We relieve the purchaser of every detail and turn the complete unit over to him under actual operation and with guarantee.

Fourth. — Every unit installed to date has exceeded its guarantee and has been completely accepted.



Remodeled Pumping Station at Bridgewater, Mass., completely installed by F. A. Mazzur Co., Inc. Barbour & Dixon, Eng'r's.

F. A. Mazzur Co., Inc.

141 MILK STREET, BOSTON

MINERALEAD

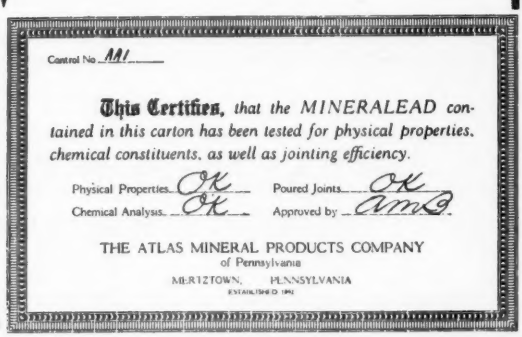
A PROVEN JOINTING MATERIAL FOR WATER MAINS

ABSOLUTE UNIFORMITY

Easy Melting
Rapid Pouring
No Caulking
Tighter Joints
No Radial Cracks

Is checked systematically to insure perfect performance.

Write for Catalogue


Control No. AA1

This Certifies, *that the MINERALEAD contained in this carton has been tested for physical properties, chemical constituents, as well as jointing efficiency.*

Physical Properties OK Poured Joints OK
Chemical Analysis OK Approved by AMG

THE ATLAS MINERAL PRODUCTS COMPANY
of Pennsylvania
MERTZTOWN, PENNSYLVANIA
ESTABLISHED 1914

New England Distributor
FRED R. HOUDLETTE & SON, INC.
40 Central Street
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STARKWEATHER & BROADHURST
INCORPORATED

Engineers and Contractors for
Water Works and Sewage Pumping Plants

79 MILK STREET, BOSTON HANcock 4530

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CAST IRON PIPE

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NEW YORK CITY

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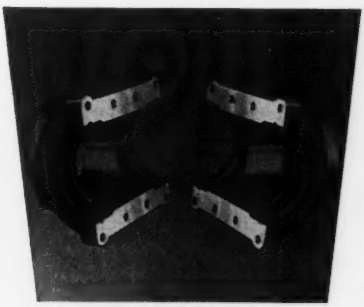


TWO BRANCHES THE SAME POINT.

SOMETIMES it is desirable to connect two new branches to an existing water main at the same point, one on either side of the latter. In such cases the Smith-Hackensack Cross Sleeve is used with the Smith Tapping Machine, without turning off the pressure and what seems to be a complicated job is easily handled with neatness and dispatch.

Send for Literature.

SMITH
WATER WORKS SPECIALTIES & SUPPLIES
THE A.P. SMITH MFG. CO. *East Orange, N.J.*

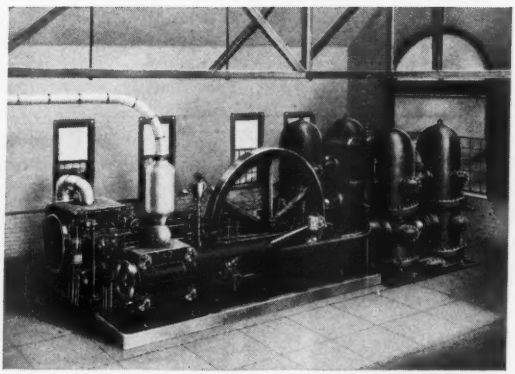
YOUR WATER METER SET IN THE
FORD
YOKE
FORD M. B. CO. WABASH IND.
May be easily and quickly removed or replaced without disturbing service pipes —— which are always connected and held in perfect alignment and spacing.
May include an inverted, ground-key stop, which saves cost of curb valve and box. Saves meter couplings, pipe fittings and pipe joints. In basement settings yoke ends may be turned to connect directly with piping. Write for catalog.
The FORD METER BOX CO.
WABASH, INDIANA

MURRAY IRON WORKS CO.

(INCORPORATED 1870)

BURLINGTON, IOWA, U. S. A.

Builders of High Duty Pumping Engines

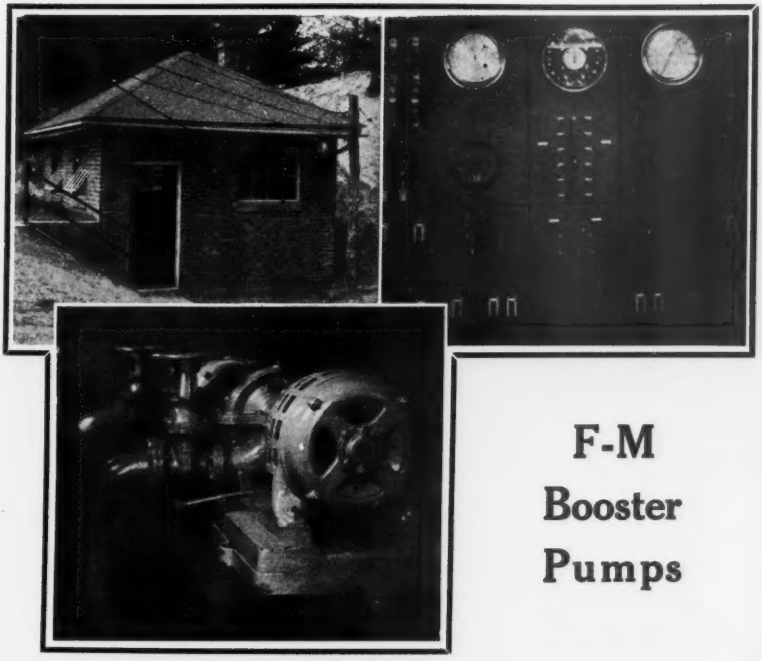


Murray High Duty Crank and Fly Wheel Pumping Engine, Opposed Type

STARKWEATHER & BROADHURST, INC.,
79 MILK STREET, BOSTON

will be glad to show you the superior design and excellent workmanship of the new 3 M. G. D. Murray Pump in the Arlington Pumping Station of the Metropolitan District Commission. Exceptional operating results are being obtained as shown by the official duty test which exceeded the guaranty by a substantial amount.

PLEASE SEND US YOUR SPECIFICATIONS AND CALLS FOR BIDS



F-M
Booster
Pumps

At SOUTH HADLEY, MASS.

Shown above are the exterior of the South Hadley Water Works Pumping Station, the two F-M Motor Driven Centrifugal Pumps installed in the station and the control equipment. The station is located at one extreme end of the town and takes water from a small reservoir under a positive head of approximately 12 or 15 feet and pumps it through the main section of the town to a standpipe at the other end. Both pumps and motors are of ball bearing design, which construction was selected because of the desirable feature of lubrication. The bearings require attention only once a year.

FAIRBANKS, MORSE & CO.

88 High Street, Boston

FAIRBANKS-MORSE

PUMPS · DIESEL ENGINES · MOTORS

DE LAVAL

Centrifugal Pumps for Automatic Operation on Driven Wells

HANOVER WATER WORKS, HANOVER, MASS.

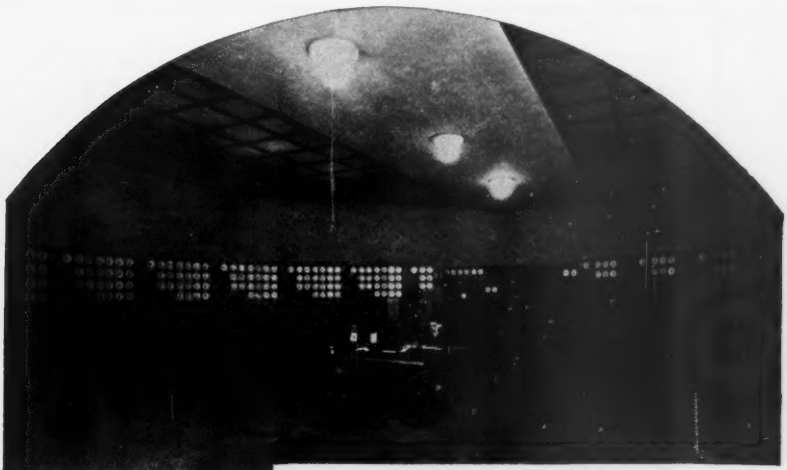
This station operates on driven wells.

Requires only a FEW MINUTES INSPECTION per day and has the following AUTOMATIC features:

1. Starts and stops from pressure governor or time clock.
2. Automatic priming with duplicate air pumps.
3. Protection against main pumps starting unless fully primed.
4. Protection against flooded pump room.

Gasoline engine auxiliary drive in case of failure of current.

TURBINE EQUIPMENT COMPANY
OF NEW ENGLAND
80 Federal Street, Boston Phone LIBerty 5993
New England Representative for
DE LAVAL STEAM TURBINE COMPANY
Trenton, N. J.



DOLLARS at stake dictate the use of these elaborate control devices on the 432,600 H. P. output of the Niagara Power Plant

CONTROL—
THE ESSENCE OF
CHLORINATION

LIVES at stake demand that chlorination, whether for drinking water sterilization, swimming pool disinfection or sewage treatment be adequately controlled.

Appreciation of this responsibility has prompted the installation of over 3000 W&T Vacuum Chlorinators to date. Their accurate control — their unquestioned dependability are a byword in fields of Public Health and Sanitation.

Radically different in principle, the hydrostatic vacuum does the work of troublesome chlorine pressure reducing devices and check valves, W&T Vacuum chlorinators are trouble free chlorinators.

NO VACUUM CHLORINATOR HAS EVER WORN OUT

Ask for Technical Publication No. 38

"The only safe water is a sterilized water"

A Product of

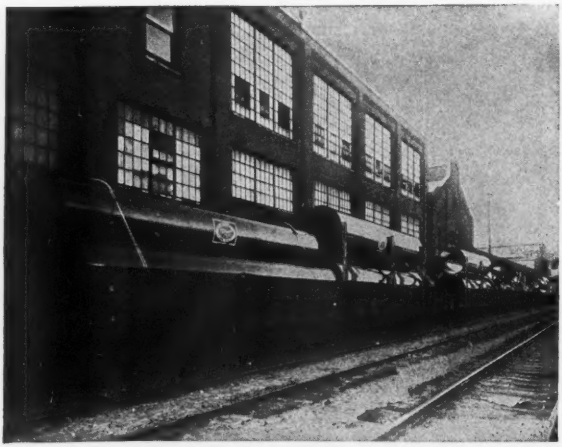
WALLACE & TIERNAN
COMPANY, INCORPORATED

Manufacturers of Chlorine Control Apparatus

NEWARK - - - NEW JERSEY

Branches in principal cities

Small SA-29



The illustration shows a partial shipment of 30", 48" and 72" diameter Biggs Electrically Welded Steel Pipe (50,000-ft. contract) in transit to Detroit, Kansas City, and Monroe, Michigan.

Biggs Welded Steel Pipe

Insures Maximum Safety

BIGGS WELD — DUCTILE — UNIFORM — STRONGER THAN THE PLATE ITSELF

WATER-WORKS engineers everywhere are placing more and more steel pipe in the ground each year because of: 1. Its dependability. 2. Its long life. 3. Its superior structural strength and ductility over all other types. 4. Its low first cost and maintenance. 5. The continuity of service and freedom from rupture which it assures.

Biggs Electrically Welded Steel Pipe represents that medium at its best, because: 1. It is manufactured under a welding procedure which controls all variables and predetermines definite results. 2. Its 100% joint efficiency has been assured by countless bend, tension, torsion, and hydrostatic tests which are impossible with any other type of steel pipe and which have clearly demonstrated that the *Biggs weld is stronger than the plate itself.* 3. Its smooth interior yields a maximum carrying capacity. Average "C" of 139. 4. The seams are welded automatically from both sides, producing a uniform, flexible, balanced weld. 5. It is furnished in long lengths — from 30 to 36 feet without intermediate seams.

Data will be cheerfully furnished upon request

THE BIGGS BOILER WORKS COMPANY, AKRON, OHIO

New York Detroit

BIGGS Steel Pipe *Riveted and Welded*

The Biggs Boiler Works Co. — Akron Ohio — Est. 1865

CAST IRON PIPE

Bell and Spigot—Sand Cast—Modern Weights

Sizes: 1¼ through 12 inches. Lengths to 16 feet

Boston stocks. Furnished with or without Precalked Joints

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Eastern Office:
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FRED A. HOUDLETTE & SON
40 Central Street, Boston
Eastern Mass., Me., N. H.

General Offices: BIRMINGHAM, ALA.
Foundries: BIRMINGHAM, ALA., PROVO, UTAH

Main's Hotcoating for the inside of standpipes and water tanks, will stop all pitting and corroding, and is guaranteed for seven years.

Also

Main's Coverall Salt Air Paint, for the outside of standpipes and water tanks, guaranteed for four years.

M. B. MAIN **Box 189, Somers Point, N. J.**

The "COREY" Hydrant

Why you should use GENUINE COREY HYDRANTS

Water to Nozzles quicker than other Hydrants.

Standpipe can be removed and replaced without shutting off water in main.

Ease of operation insured by knuckle joint principle.

Only 13 turns to completely open or completely close, but full flow with only 4 turns.

Nozzles screwed in — not leaded.

Can be lengthened without shutting off water.

Will not flood if broken.

Positive Drip — cannot freeze.

Standpipe can be revolved quarter turns — Nozzles always in right direction.

Write for copy of our new Catalog "G."

CHARLES L. BROWN,

New England Sales Manager, Lock Box 2, Northboro, Mass.

RENSSELAER VALVE COMPANY

TROY, N. Y.

New England Water Works Association

Specifications, Forms, and Meter Rate Sheets

Standard Specifications for Cast-iron Pipe and Special Castings,	per copy,	$0.25
Forms for Slow-sand Filtration Plants	per set,	0.30
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100 sheets, in orange		2.50
100 sheets, in black		2.25

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WATER & GAS PIPES

FLANGE PIPE

Special Castings, Fire Hydrants, Valves

General Foundry and Machine Work

WOOLWORTH BUILDING 233 BROADWAY

NEW YORK CITY

HIGHEST AWARD, GOLD MEDAL,
ST. LOUIS EXPOSITION, 1904

Over 100 Water Departments

use

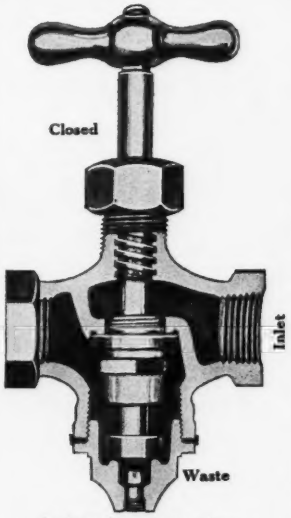
LEAD-LINED IRON and TIN-LINED PIPES

for their service connections

MANUFACTURED BY

Lead Lined Iron Pipe Company

Wakefield Mass.



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Copper Flanged Corporation Stop

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Byers' Wrought Iron Pipe
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SUCCESSORS TO
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WHOLESALERS OF

Steel, Wrought Iron, Brass Pipe and Water Works Materials of all kinds including copper service pipe and fittings.

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CEMENT LINED SERVICE PIPE

With Specially Adapted Lead-lined Fittings

Also Curb Cocks which eliminate Iron to Brass Corrosion both inside and out.

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LYNN, MASS.

CLARK METER BOXES

Cover and Lid Type

Perfect Lock
Wonder Lock
Worm Lock

Permacrete
Gray Iron
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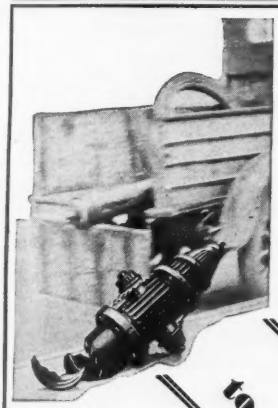
Armco Iron

Investigate Permacrete Now

H. W. CLARK CO.
MATTOON, ILLINOIS



MATHEWS MODERNIZED HYDRANT

Reg. U. S. Patent Office

No digging or breaking the pavement. Screw out the old—screw in the new—as easy as changing a tire!

A broken Mathews cannot leak

20 MINUTES—to replace a broken Mathews

Read this letter from Bedford Hills, New York, where these photos were taken.

"With your tubular lug wrench the broken barrel was easily removed, and the new barrel installed, the nozzle direction set by means of the swivel flange, and the whole job completed *in twenty minutes without disturbing the cement sidewalk* in which this hydrant was set.

"We certainly do appreciate this quick-replacement feature of the MATHEWS, especially considering the labor and repaving costs that it saves; and we salvaged the top cap and nozzle section for later use."

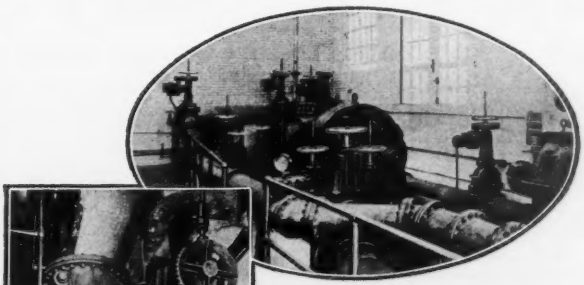
Write for the new MATHEWS MODERNIZED HYDRANT BOOKLET that tells the whole story.

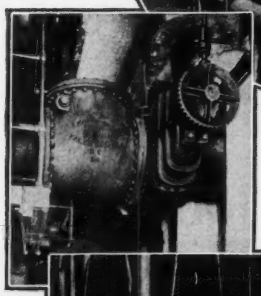
R. D. WOOD & CO.

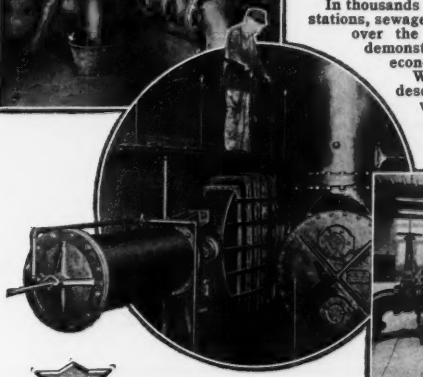
In business continuously since 1803

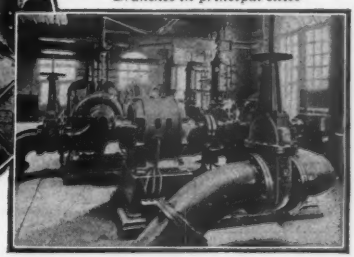
400 Chestnut St. Philadelphia

CAST IRON PIPE, SAND SPUN (centrifugally cast) and PIT CAST Reg. U. S. Pat. Off.
GATE VALVES VALVE BOXES FLOOR STANDS









No matter what the control

Kennedy Valve operation is always dependable

NOTICE the many different types of control in these views of typical Kennedy Valve installations. Electric gear, extended stem, hydraulic, and manual control are all shown — AND IN EACH CASE THE SERVICE IS THOROUGHLY SATISFACTORY.

This is not merely because the controls alone have been carefully planned and adapted to the specific requirements of each installation, but further, because each Kennedy Valve itself is efficiently designed, honestly built, and ideally suited to the service for which it is offered.

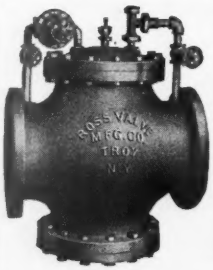
In thousands of municipal water works, pumping stations, sewage plants, and fire protection systems all over the country, Kennedy Valves are daily demonstrating their reliability, efficiency and economy of operation.

Write for the Kennedy catalogue describing the complete Kennedy line of valves, pipe fittings, and fire hydrants.

THE KENNEDY VALVE MFG. CO.
Elmira, N. Y.
Branches in principal cities

KENNEDY

VALVES~PIPE FITTINGS~FIRE HYDRANTS



THE STANDARD FOR 50 YEARS

Automatic Pressure Control Valves
Feed Water Filters
Portable Fire Hydrants
Hydraulic Engines
Hydraulic Engines—Booster Pumps

ROSS VALVE MFG. CO., INC. **TROY, N. Y.**

Health and Protection First

Water delivered through dirty pipes may be a MENACE.

Incrusted water pipes mean inefficiency and loss of Fire Protection.

We Guarantee the Results of Our Method of Cleaning.

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National Water Main Cleaning Co.

50 Church Street **New York City**

HAYES PUMP AND MACHINERY CO.

94 PEARL ST., BOSTON, MASS.

Pumping Plant Contractors

CENTRIFUGAL, POWER, STEAM PUMPS

STEAM TURBINES
WATER WHEELS

ELECTRIC MOTORS
OIL ENGINES

GOLDEN-ANDERSON

Patent Automatic Cushioned Controlling Altitude Valves

We have enlightened many a water user in municipalities, in industrial plants and on railroads, as to the adaptability of these Golden-Anderson Valves, and as to their function of SAVING, and of eliminating troubles.

For automatically maintaining Uniform Stage of Water in Tank, Reservoir or Standpipes. Doing away with the annoyance of Freezing and Float Fixtures inside or outside of Tanks.

"Three Ways of Closing These Valves":

1st —Automatically, by water.
2nd—By Electricity, if desired.
3rd —By Hand

May also be arranged to automatically close when a break occurs in the mains. When necessary they may be so connected as to "work both ways" on a single line of pipe. No valves or fixtures inside or outside.

Sizes to 24 in.

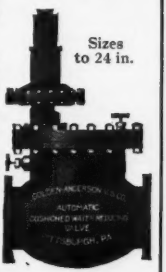
Remember!

"HOSTS OF REFERENCES"
Valves cushioned at all times by air and water
No water hammer or bursting mains

"Made with stop starter attachment for centrifugal pumps"

GOLDEN-ANDERSON
Pat. Automatic Cushioned Water Pressure Regulating Valves

1. Maintain a constant reduced pressure regardless of fluctuations on high pressure side.
2. Perfectly Cushioned by water and air. No metal-to-metal seats.
3. The best valve made for maintaining a constant low pressure where consumption is continuous.
4. Operates quickly or slowly as required — No attention necessary.
5. Positively no hammering or sticking.



Sizes to 24 in.

GOLDEN-ANDERSON
Patent Automatic Cushioned Controlling Float Valves

They carry a constant water level in feed water heaters by perfectly controlling the flow of makeup water.

Operated by protected enclosed copper float.

Cushioned by both air and water.

Operate without hammering, sticking or chattering. No metal-to-metal seats. Made angle or straightway.

Sizes 1 in. to 24 in.

GOLDEN-ANDERSON
Pat. Automatic Cushioned Water Float Valves

1. Automatically Maintain Uniform Water Levels in Tanks, Standpipes, etc.
2. Instantly Adjusted to Operate Quickly or Slowly.
3. Floats Swivel to any Angle — Most Satisfactory Float Valves Known.
4. No Metal-to-Metal Seats — No Water Hammer or Shock.
5. Cushioned by Water and Air.

Angle or Globe

Sizes to 24 in.

GOLDEN-ANDERSON
Patent Automatic Double Cushioned Check Valve

Sizes to 24 in. Angle or Globe

Automatically prevents reverse flow of pressure.

Perfectly cushioned to prevent shock or hammer.

Especially adapted for water works to prevent disastrous reverse flow of pressure due to pumps stopping.

Arranged so that quick or slow operating may be obtained.

Golden-Anderson Valve Specialty Co., 1325 Fulton Bldg., Pittsburgh, Pa.

-and now

Introducing!

The New PARADON

Solution Feed Chlorinator

The new PARADON Solution Feed Chlorinator is furnished in capacities ranging from .03 to 360 pounds of chlorine per 24 hours. It is of semi-vacuum type, part of the apparatus being under a slight positive pressure and part being under a slight negative pressure.

An entirely new injector is used. Some of its features include: extremely low water consumption, interchangeable throat, adjustable water supply, check valve to prevent flooding of the chlorinator. The whole unit may be quickly dis-assembled for inspection. It represents a big step forward in solution feed chlorination.

The experience gained in the manufacture and installation of over 1200 chlorinators during the past seven years is behind this new PARADON unit.

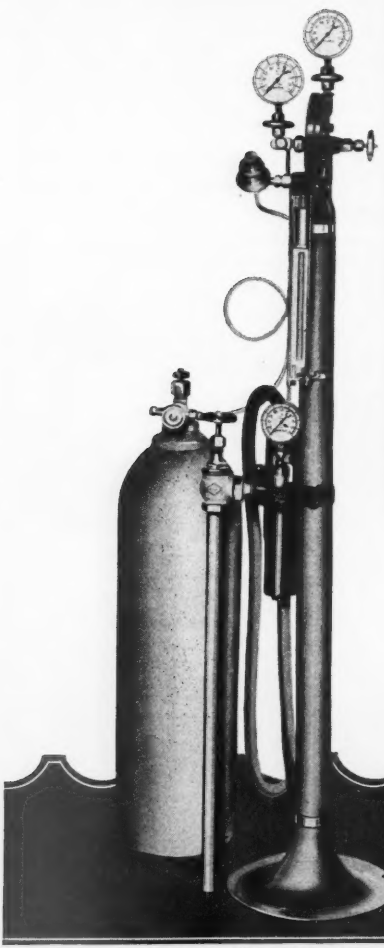
Send for Bulletin N-27.

THE PARADON COMPANY

Arlington, N. J.

Paradon CHLORINATORS

For City and Private Water and Sewage Systems, Industrial Plants and Swimming Pools,



All Paradon Solution Feed Chlorinators are licensed under the Ornstein Patent.

N22sn.

Type of Eddy Hydrant Popularly Used

More Than Sixty Years of Experience Goes Into All

EDDY

Hydrants and Valves

BUILT TO MEET EVERY EXACTING REQUIREMENT OF WATER-WORKS SERVICE

Practical water-works men instantly recognize the superior features of Eddy products. For instance, with the hydrant.

A damaged valve can be replaced in a few minutes — Water hammer is avoided by the valve closing against the pressure and the use of the cut-off under the valve, which gradually closes it —
Stems cannot be bent in closing the hydrant.
Nothing can lodge on the valve seat to prevent closing of the valve —
The Drip Rod can be removed without interfering with the hydrant in any manner —
The position of the stem of the hydrant indicates accurately the position of the valve —
Whatever can pass the valve opening will pass the standpipe —
No frost case necessary —
All of these points are explained thoroughly in our catalogue, which will pay you to study.

The Eddy Valve, Too

Has its distinctive and individual features. The double disk gates are free to adjust themselves in different positions every time the valve is closed, thereby keeping smooth and tight faces. And two bronze hooks on the ball loosely engage with the gates to prevent them from spreading at the top. The center bearing gates are forced to their seats with equal pressure at all points. There is a good deal more worth knowing about Eddy Valves. Further details will be sent for the asking.

Other Eddy Products

The Eddy line of water-works products also includes horizontal and vertical check valves, foot valves, hydraulically operated valves, indicator posts, etc.
All goods made by the Eddy Valve Company are manufactured exclusively at Waterford, New York.

WRITE FOR CATALOGUE

EDDY VALVE COMPANY

WATERFORD, N. Y., U. S. A.

DONALDSON IRON CO.

MANUFACTURERS
...OF...

CAST IRON PIPE

...AND...

Special Castings for Water and Gas.
Also Flange Pipe and Fittings.

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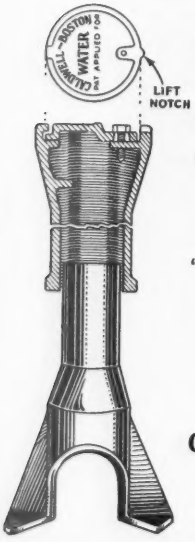
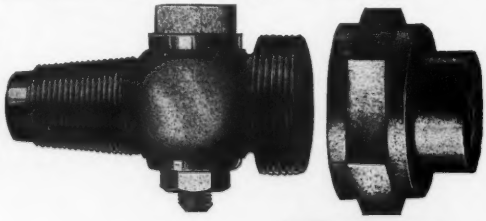
LEHIGH COUNTY, PA.

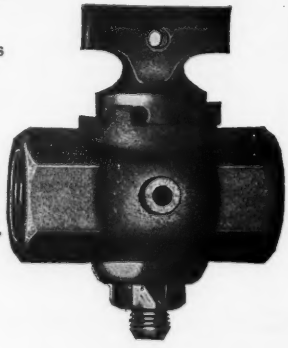
The "BEST" Cast Iron Curb Box

Top section is interchangeable with the "Slide Type" Buffalo Box



No change necessary in wrench or shut off keys





"CALDWELL" Water Works
Brass Goods, Curb and
Valve Boxes

None better and few as good

GEORGE A. CALDWELL CO.
Mattapan Square
BOSTON, MASS.

National-Boston Lead Co.

800 Albany St., Boston

Agents for

The Celebrated "ULCO"

LEAD WOOL

(Every Atom Pure Lead)

For calking pipe joints under the most difficult conditions.

For overhead joints, or in wet places where the use of molten lead is not only impracticable but dangerous — **LEAD WOOL** may be used to advantage.

It makes an absolutely tight joint which will withstand the highest pressure, yet be sufficiently elastic to allow considerable sagging or settling of the pipe without danger of a leak.

As compared with the poured joint, in this respect, the superiority of **LEAD WOOL** is apparent.

Manufacturers of

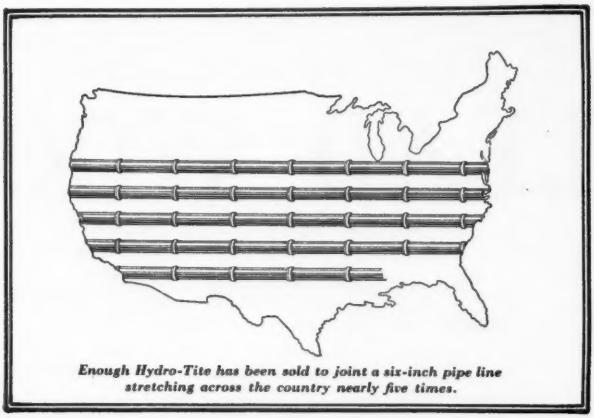
Lead Pipe, Tin Lined Lead Pipe, Pure Block-Tin Pipe, Solder, White Lead and Red Lead.

HIGHEST QUALITY SOFT BRANDS

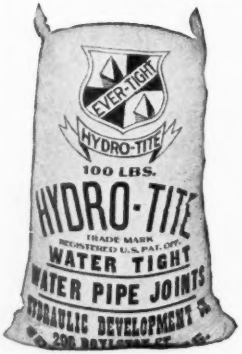
PIG LEAD

HYDRO-TITE

— *its record . . .*



Enough Hydro-Tite has been sold to joint a six-inch pipe line stretching across the country nearly five times.



HYDRO-TITE is always the same, comes in moisture proof, non-dusting bags, is shipped the day order is received. Every bag is subject to a guarantee which makes it impossible for you to lose by trying this modern pipe jointing material.

. . . over 21 years without a failure

Millions of joints in cast iron bell and spigot water pipe have been made with HYDRO-TITE.

In over 21 years not a single failure has been reported to us.

No wonder that year by year more waterworks men specify and use HYDRO-TITE and HYDRO-TITE equipment. They profit because HYDRO-TITE joints never blow out, leak 75% to 95% less than engineers specify for lead joints, withstand deflection and vibration, save 75% as compared with lead, require no caulking.

Write for the new HYDRO-TITE *data book, just issued*

Hydraulic Development Corporation

Main Sales Office:
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General Offices and Works:
80 Boylston Street, Boston, Mass.

EVER-TIGHT
HYDRO-TITE

HYDRO-TITE

Reg. U. S. Pat. Off.

TODAY'S SELF-CAULKING WATER-PIPE JOINTING COMPOUND

5237

ECONOMY is half the battle of life; it is not so hard to earn money as it is to spend it well.
Spurgeon.

To Specify Darling Valves and Hydrants is Sound Economy

The purchase of valves and hydrants should be determined by their fitness for the task — and not simply on a price basis.

When you buy Darling products you may pay a few cents more. Many companies have. And not one has experienced any regrets for the slight increase in initial cost. They have found that money spent on Darling valves and hydrants is money well spent. They know that Darling products are different. Different in the extra service rendered. Different in the matter of internal mechanical construction.

For example. The Central Bearing Point in Darling Gate Valves on the wedge faces allows the disc face to find a true and uniform bearing on the seat face — one of the features which insures tight closing, even wear and long life.

Darling Hydrants, too, have many exclusive features. At your request we will send complete information.

Darling Valve & Mfg. Co.

Williamsport, Pa.

New York Los Angeles
Oklahoma City Houston

DARLING
GATE VALVES &
FIRE HYDRANTS

For the CITIES of
TODAY and TOMORROW
The LUDLOW DIAMOND
a new self-illuminating Fire Hydrant
that keeps pace with progress

This is news—good news—to city officials, water department engineers, fire chiefs. Be sure you get a copy—we have one for you, free. Write for it today.

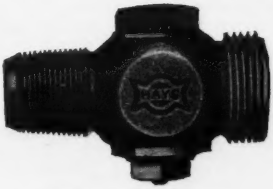
Have you received your copy of this new folder?

This hydrant is *news*—good news—to city officials. Its handsome design is in keeping with the newer architecture and the improvements in modern street lighting. Its octagonal dome and caps provide extra light-reflecting surfaces that mean quick pick-up—saving precious seconds. Its three-part construction permits replacement without digging—cuts costs, lowers inventories. And its improved valve mechanism will not flood even when the hydrant is broken off.

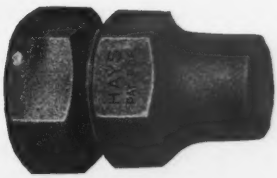
A cost of less than 2¢ per unit per year for repair parts, is the record established during half a century by 8,000,000 Ludlow Valves and Hydrants in over 3,000 cities and towns, proving that Ludlow quality insures *lasting* economy.

The TRADE LUDLOW MARK VALVE MFG. CO.
TROY · NEW YORK
MAKERS OF *DEPENDABLE* VALVES AND HYDRANTS SINCE 1866

The Hays Copper Plumbing Method

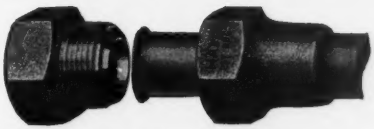


Regular Corporation Stop



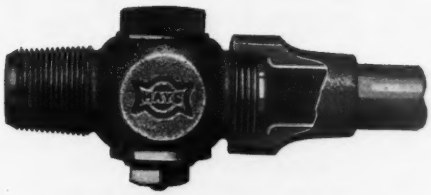
Adapter

In making an installation with Hays Copper Service Fittings the regular Corporation Stops can be used in conjunction with the adapter. It is not necessary to procure new mandrels, as the Corporation Stops are inserted with the regular mandrels, in the usual manner, after which the adapter is attached.



Open View of Adapter

These adapters are made with Female Thread for Increased I. P., Water Works Standard or Lead Flange.



To use standard Corporation Stops with Increased I. P. or Water Works Standard Thread, and where a direct connection of the Corporation Stop and Copper Pipe is desired, the above stop has been designed. (See cut above for sectional view of joint.) These stops permit a Hays 90° seal with the use of regular male mandrels.

Write for complete details of

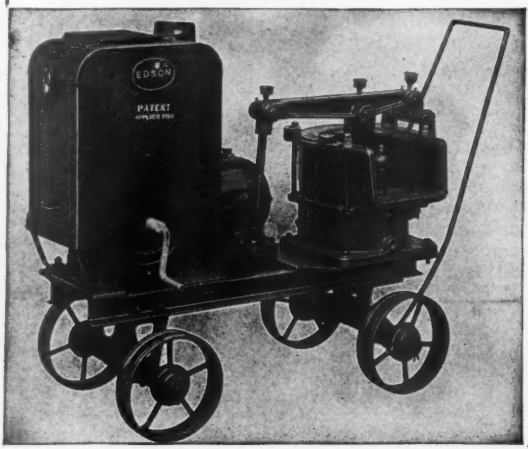
The Hays Copper Plumbing Method for Water Works and House Plumbing

Hays Manufacturing Co. Erie, Pa.

EDSON MFG. CORP.

DIAPHRAGM PUMP SPECIALTIES

Hand Pumps. Heavy Duty and Light Portable Power Pump Outfits



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Special Suction Hose

Red Seal Diaphragms

Pump Accessories

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SALES OFFICES

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75 FEDERAL STREET, BOSTON, MASS.

Manufacturers of

CAST IRON PIPE

Flanged Pipe *Flexible Joint Pipe*

Bell and Spigot Pipe

Special Castings *Short Body B. & S. Specials*

Warren (WF) Spun Centrifugally Cast Iron Pipe

WORKS: PHILLIPSBURG, N. J. and EVERETT, MASS.

Large Stock Enables Us to Make Prompt Shipments

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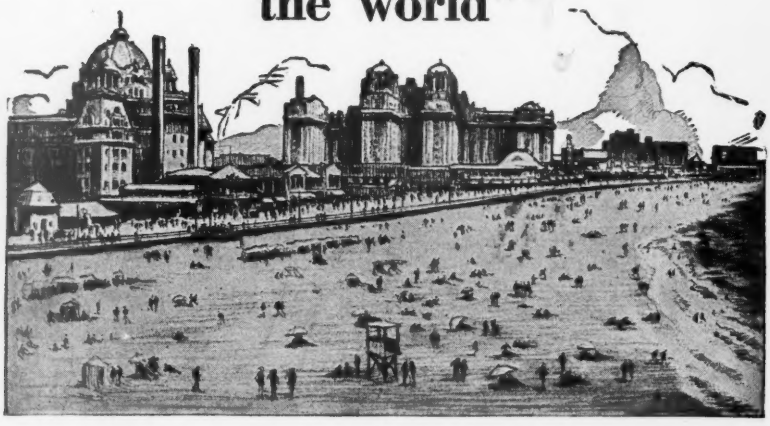

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